DETERMINATION OF SIDE FRICTION IN DRILLED SHAFTS SOCKETED IN ROCK USING PULLOUT TESTS

BY

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A DISSERTATION PRESENTED TO THE GRADUATE SCHOOL
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To my parents who taught me how to respect God and society and gave me a solid education. To all my teachers.
To my family who keeps me going with optimism.
To my brother, to whom I could not say good bye.

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by

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Chairman: Dr. Frank C. Townsend
Major Department: Civil Engineering

Previous research conducted at the University of Florida has shown that conventional soil/rock exploration data, namely penetration test blow count and recovery indices, do not provide reliable guidance in assessing either the friction or the end bearing capacities for drilled shafts socketed in rock. The literature search indicated that current design methods are essentially empirical and therefore should be verified for Florida conditions.

The research focused on a technique that can be used to assess the maximum lateral (side) friction shear stress along the rock socket. A small scale anchor--cast using a fluid grout--is pulled out in order to define the maximum side shear. The movement of the anchor can be monitored so that the T-Z curve can be obtained.

Thirteen tests were performed at three sites using various new techniques to cast the plugs and measure their

displacement. In addition, information was obtained from pullout tests completed in the Miami Oolite formation.

An attempt was also made to use nondestructive lab seismic tests on the rock cores.

The main conclusions can be summarized as follows:

The anchor length/diameter ratio may affect the computed maximum side shear, at least for the types of rocks investigated. More tests are required to clarify this point.

It appears that the initial portions of the T-Z curves found from the pullout tests and those from full size load tests, seem to agree.

The displacement required to mobilize the maximum side shear fell between 0.1 and 0.2 inches. This range is also similar to that found in full size load tests in previous research.

The obtained Alpha values fell either close to or above the lowest of the existing guidelines, thus indicating that this guideline could be appropriate as a starting point for preliminary design.

The evidence seems to support the new predictive method proposed by McVay.

The results indicate that a relationship exists between the P-wave speed and the unconfined compressive strength of the rock cores.

The P-wave speed appears to be related to the secant slope of the T-Z curves, at maximum side shear.

CHAPTER 1
INTRODUCTION

During the past four years, the UF Department of Civil Engineering (Geotech Division) has been involved in research on deep foundations for the Florida Department of Transportation (FDOT). Both driven piles and drilled shafts installed in soil and/or rock have been considered. One of the purposes of the investigation has focused on building a data base so that the predictions of axial load capacity and settlement according to current design methods can be compared to either actual shaft behavior or to miniature piles modeled in the centrifuge. Another area, in-situ testing, has also received special attention.

This report complements existing methodology of a technique used to obtain the lateral side friction of drilled shafts socketed in weak limestones, namely the pullout test (Figure 1.1). In essence, the method consists of casting an anchor (usually two to six feet long) at the desired depth, waiting for the grout to set (commonly three to five days) and then pulling the plug until the resisting force developed in the rock is overcome. The technique and the required setup are quite simple, thus permitting one to perform several tests in a given area. Although the basic idea was proposed several years ago by Schmertmann (1977), it seems that it has only been used on

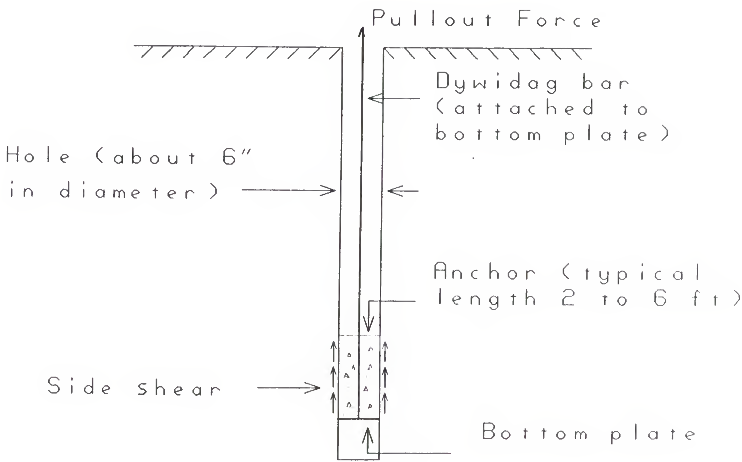


Figure 1.1. Pullout test basic idea.

a regional basis in Florida, primarily in the southeast area (Miami and the Keys), and in the Tampa area. It should be pointed out that the procedure to install the grout plugs and to measure the displacement during the test is easier now than when Schmertmann made his proposal. Commercial grouts that flow quite well, thus facilitating the pouring process, and which yield high strengths within a few days, are now available. Also, simple electronic devices to measure displacements and pressures accurately are available, as well.

The procedure to drill the hole, install and test a plug (or anchor) does not require a long time. One or two days are needed to drill/case a hole to a depth ranging between 40 and 60 ft. The grout requires about twenty minutes for placement and three to five days for hardening. About two hours are spent in performing the load test. The required personnel (excluding the drilling crew) does not amount to more than three to five people. Although no economic analyses were performed, it appears that the cost would be much less than that involved for a load test on a full size shaft. Several pullout tests could be performed <u>at different locations and depths, thus getting a better coverage of the site,</u> for the cost of a single load test. However, the full size test has a distinct advantage in that both the construction procedures and the scale factor are directly taken into account.

Experience collected throughout the study indicates that the usefulness of the results obtained from load tests on instrumented shafts depends on several factors, among which the following seem to be the most relevant: the type of instrumentation (sisterbars are to be preferred, telltales seem to be less reliable); the disposition of the instruments (it is especially important to locate several in those areas where either the geometry of the shaft or the type of rock/soil change); and the presence of a casing, commonly used to avoid friction in the upper layers. Depending on the way the casing is installed, the purpose may not be accomplished fully, thus complicating the interpretation of the results. Although a successful test yields valuable data, it may represent a <u>very limited portion of the construction area</u>. The extrapolation to the design of other shafts may involve a much less refined (and somewhat more subjective and less documented) approach in order to account for the spatial variability, e.g., the use of drilling records (times and rates of advancement), recovery indexes (RQD, for example), records of strength of the rock cores, etc.

It must also be recognized that, in most cases, the available load test setups (maximum capacity of about 1200 tons) <u>do not permit the loads required for full mobilization of the side friction along the entire socket to be imposed</u>. On the other hand, a simpler reaction system and a 100-ton jack are, in most cases, enough to find the

maximum side friction via a pullout test at any depth of interest.

The literature search indicated that available design methods for rock sockets are essentially empirical, and therefore valid strictly for the conditions under which they were developed. Any such guideline should be validated or adjusted for a given site.

The reasons exposed previously may indicate that the research being reported is needed to improve the design methodology of rock sockets.

The present investigation concentrated entirely on side friction behavior. The tip component has been temporarily discarded because, as theory and some load test results indicate, it represents a small fraction of the total load capacity (maximum 20% at maximum load at the top and less than 10% under working conditions).

A series of pullout tests were performed and a program of nondestructive and destructive tests carried out on recovered rock cores to verify and complement some well known correlations between the maximum side friction and the strength of the rock. An attempt was also made to set points of departure for other types of correlations not found in the available technical literature.

One basic assumption was adopted in regard to the diameter of the plug (anchor), namely that the scale effect (difference in the side friction of a small anchor and a full size shaft) could be disregarded provided the

diameter of the former exceeded about 5 inches. The hypothesis seems to be substantiated by experimental data presented by Horvath and Kenney (1979) and more recently by Horvath and Chae (1986). One of the tests performed for this study also seems to verify the assumption and will be presented later in the report.

The considerations just presented led the research team to choose a 4-inch core barrel to drill the holes. The device allows one to recover 4-inch diameter rock cores (nominal size) and yields a hole with a nominal diameter of 5.5 inches. Such a large core size has the advantage that the recovered pieces are more likely to have natural fissures, not present in smaller cores (e.g., the NX). Thus, stronger and weaker materials are sampled.

In order to compute the side friction one needs to know the dimensions (length and diameter) of the anchor. Some researchers and practitioners strongly recommend that the plug be recovered once the test is completed. It must be realized that the recovery process complicates the setup to some extent (a crane able to apply several tons may be required in some cases). Alternatively, one could measure the diameter of the hole prior to casting the anchor, keep a record of the amount of grout poured and measure the elevation at the top of the anchor just before performing the pullout. Both approaches were used in this research, as will be shown later.

It must be pointed out that practically all the tests were performed in holes drilled without any stabilizing agent (mud). Drilling mud was used in one location only, so the effect of this variable cannot be assessed reliably. The possible effect of the type of tool used to drill the hole was not studied, either.

CHAPTER 2
PURPOSE AND SCOPE

The study evolved around the following aims:

A) To complement previous experiences on the use of the pullout test technique to define maximum side friction in some typical weathered calcareous rocks present in Florida.

B) To validate and complement existing guidelines, namely those proposed by McVay and Townsend (1991), Williams et al. (1980a), Rowe and Armitage (1987), and Gupton and Logan (1984), for the case of the calcareous Florida rocks. The guidelines are based on correlations between the "intact" strength of the rock obtained from unconfined compression and/or split tensile tests and the expected maximum side shear.

C) To innovate a technique to obtain the so-called T-Z curves from the pullout test. The curves can be input into existing computer codes (Load-Transfer approach) and the Load-Settlement of a full size shaft may be computed. With such information, a decision can be made about the load associated with a "tolerable" settlement.

Traditionally, the Ultimate or Maximum load, i.e., the sum of the maximum side friction and the tip component (if reliable information is available with respect to end

bearing behavior), is divided by a factor of safety (usually between 2 and 3) to get a safe (or working) load. This working load is used to compute the settlement utilizing solutions obtained from elastic theory (usually specialized for homogeneous and isotropic medium).

Although the basic idea looks simple, it requires that the modulus of elasticity of the mass be known or assumed. This step is somewhat difficult to perform at this time because little information is available in the literature about the deformability characteristics of the intact rock, and very little appears to be known about the reduction that should be introduced to account for discontinuities in Florida rocks.

Therefore, the use of the load transfer approach emerges as a reasonable alternative. The T-Z curves obtained from pullout tests involve the response of the rock mass (at least to some extent) thus circumventing the difficulties just presented. On the other hand, the test seems simple enough so that the site can be covered and spatial variability taken into account.

D) To attempt to set points of departure for correlations between the results of nondestructive tests (e.g. lab seismic tests) and the results of the pullout tests.

The research involved the following tasks:

A) Collecting data on pullout tests performed in Florida.

B) Performing a literature review.

C) Performing a series of tests to choose the grout.

D) Designing and building a caliper to measure the diameter of the holes.

E) Performing the pullout tests.

F) Pursuing the lab testing program on the rock cores recovered at the different sites.

CHAPTER 3
BACKGROUND AND PREVIOUS RESEARCH AT U.F.

Summary of Research Steps on Rock Sockets in Florida

The first steps of previous research (Parra et al., 1990) consisted of the following items:

A) Compiling a database of load tests on instrumented shafts socketed into rock. The database included a series of T-Z curves and the results of the in-situ exploration (standard penetration test, almost exclusively), as well as some information about the strength (unconfined compression) and the elastic modulus of the Florida calcareous rocks.

This step included an analysis of the reliability of one of the schemes commonly used to instrument the shafts, namely the telltales; computer codes were devised to reduce the data, including the case of the sisterbars.

B) Conducting a literature review on the current methods for designing drilled shafts socketed in rock and on the geology of Florida.

C) Performing the geotechnical exploration of a research area (Kanapaha site, located in Gainesville). Several penetration tests (SPT and cone soundings) were performed to define an area in which a socket could be

installed. A pullout test was also conducted at the chosen location. The results of the program just presented were used to design the shaft, following the FHWA indications for the overburden and Williams et al. (1980a) alpha method for the weathered rock.

D) Constructing and load testing a full size (24 inch diameter) shaft socketed into very weathered rock (Kanapaha site). The shaft was thoroughly instrumented by means of sisterbars, foil strain gages attached to the rebar cage, and telltales. A low strain dynamic test was performed to inspect the integrity of the shaft, followed by a dynamic test to define load capacity. The top of the shaft was hit with a 10-ton hammer free falling from heights between 3 and 8 ft, and the collected data (records of stress and velocity near the top) were input into the CAPWAC package. Finally, a conventional static quick load test was conducted (Townsend et al., 1990).

Summary of Findings and Conclusions

Rock Characteristics

The main conclusions (Parra et al., 1990) can be abridged as follows:

A) Geologically, there are eighteen calcareous formations in Florida. The rocks show mechanical properties that may vary quite noticeably both in the vertical and the horizontal directions.

B) The characterization of rock for foundation engineering purposes is usually done by means of the

Standard Penetration Test, the recovery indexes (especially the RQD) and the strength of intact cores (usually by unconfined compression). Strengths fall in a wide range, from about 50 psi to more than 6000 psi, with 500 psi as a rough average. Little information was gathered about the elastic modulus. Available data indicate that it falls within the range, 400,000 to 2 million psi.

The collected information on RQD shows that, in general, the rock quality can be considered "fair" at best, according to Deere's criterion. It is interesting to note that no information seems to be readily available concerning the discontinuities (joint patterns or separations) of the rock masses.

The exploration at the Kanapaha site indicated a very irregular rock surface profile. A series of "chimneys and valleys" separated by a few feet in a random pattern were detected, thus making the selection of the site for the shaft difficult.

T-Z Behavior

The collected information (Parra et al., 1990) showed that:

A) The maximum side shear varies within a wide range; the lowest recorded values are on the order of 1.5 tsf (very weathered rock) while the highest exceeded 25 tsf.

B) The vast majority of the T-Z curves display a nonlinear shape and a strain-hardening type of behavior

(without a post-peak drop in strength). This indicates that the shafts may be rough, according to the findings by Pells et al. (1980). The displacement required to mobilize the maximum friction ranged from 0.1 to 0.2 inches, but in general was less than 0.2 inches.

C) No reliable correlations could be found between the maximum side friction and the results of in-situ exploration, namely the SPT, the recovery index, or the RQD.

D) The alpha values (maximum side friction back-computed from a load test, divided by the unconfined compressive strength of the rock) tend to fall near the curves proposed by Williams et al. (1980a), Rowe & Armitage (1987), and Gupton & Logan (1984). Nevertheless, the scatter was noticeable and the predictions via either method could err on the unsafe side.

E) There is not enough information to define the possible effect of drilling mud on the T-Z behavior. Apparently, based on two cases (Parra et al., 1990), the slope of the curve could be smaller if the shaft is drilled using mud. On the other hand, several of the points located under the current design curves (Williams etc.) corresponded to shafts drilled without any mud.

F) Information about load and tip displacement of the shafts is scarce. Available data indicate that about 5% to 25% of the total load reaches the tip of the shaft for settlements of about 0.1 inches to 0.5 inches, respectively. For very short sockets, i.e., length/diameter

ratio smaller than about 2, the proportion seems to be much higher, about 70%. In any event, there is not enough information to define the relationship between the item in question and the strength and deformability of the rock (intact cores and rock mass). Two cases were compiled that indicate that the ratio of stress at the tip, at maximum test loads, to rock uniaxial compressive strength may be of the order of 2 or even larger.

About the Instrumentation

The collected experience (Parra et al., 1990) indicated that the accuracy of the telltale measurements seems to be somewhat questionable. The computed shear and tip stresses displayed unreasonable patterns in practically all the analyzed cases. Three factors can be invoked to explain the fact, namely temperature effects, the type of dial gages (commonly 0.001-inch accuracy) used, and the presence of bends, either in the telltale rod or in the casing, that impede the free movement of the rod.

The instrumentation installed on the Kanapaha test shaft performed satisfactorily. The results of shaft shortening found via telltale readings (using 0.0001-inch dial gages) and the sisterbars compared favorably. The sisterbars yield "smoother" records and should be more accurate. It must be pointed out that the number of installed telltales was not enough to define the load shedding via this type of instrumentation.

Static and Dynamic Load Testing

The shaft installed at the Kanapaha site was dynamically and statically tested, as explained before. The results (Townsend et al., 1991) indicate that both techniques lead to similar total (friction plus end bearing) load capacities. However, the friction-load/tip-load ratio, as well as the load shedding pattern along the shaft differed substantially from one method to the other.

Design Methods

A total of six methods used in various parts of the world and Florida were compiled during the literature search. They can be grouped into the following categories:

A) Empirical

METHOD SOURCE	AIMS	INPUT
C.I.R.I.A (1979)	Ultimate capacity (friction and tip loads)	SPT blow count
Crapps (1986)	Ultimate capacity (friction and tip loads)	SPT and data from load tests
Gupton and Logan (1984)	Ultimate friction	Rock unconfined compressive strength

B) Semi-empirical

METHOD SOURCE	AIMS	INPUT
Williams et al.(1980a) Rowe and Armitage (1987)	Ultimate capacity (friction and tip loads) Settlement	Rock unconfined compressive strength Elastic moduli (intact core and rock mass) Elastic modulus of concrete Shaft roughness
McMahan (1988)	Ultimate capacity (friction and tip loads) Settlement	SPT blow count

The first semi-empirical method (Williams et al., 1980a) resorts to elastic solutions to find the settlement and the proportion of the load that reaches the tip of the shaft. Based on field data, the above mentioned author proposes a way to separate the elastic (linear) and the plastic part of both the friction and the tip loads, so that the nonlinear behavior can be introduced in an approximate way in the analysis.

Rowe and Armitage (1987) use a more sophisticated approach. The settlement analysis is handled via nonlinear finite element analyses that consider the slip along the shaft-rock interface. It should be pointed out that a set of parameters obtained for two Australian rocks was used to develop the method because, as explained by the authors, these parameters are usually not available. The recipes to find ultimate loads (friction and tip

components) are analogous to those proposed by Williams et al. (1980a).

McMahan (1988) proposed guides to define T-Z curves based on the SPT and the results of load tests. The behavior of the shaft can be examined by means of available codes (load transfer approach).

It is evident that all the methods rely on local field experience and therefore their applicability, as one would expect, is limited. One must also keep in mind that some of them require basic data that, apparently, has not been determined for the calcareous rocks of Florida. The shear strength parameters (angle of friction, cohesion, dilatancy) along the shaft-rock contact, the ratio of elastic moduli of the intact rock to that of the rock mass, and the roughness of the shaft are examples. Therefore, one has to make some guesses to apply a given approach. One may as well keep in mind some other items, as follows:

A) The recipes are based on load tests that most likely were performed under different conditions, with different types of instrumentation and most importantly, in <u>different</u> rock types. On the other hand, it is somewhat difficult to trace back the original data on which each author based his proposal to assess the possible effect of the items just mentioned.

B) A similar situation arises in connection with the strength of the rock. Most likely the unconfined compression tests were performed on cores with different

diameters (thus obscuring the effect of the sample size due to cracks), and most importantly, an average value along the shaft had to be used to compute a punctual alpha value. How this average takes the spatial variability into account is completely unknown or difficult to trace back at this time.

CHAPTER 4
TASKS PERFORMED

Theoretical Backup

One of the aims of the research was to attempt to define the so-called T-Z curves so the Load Transfer Approach (LTA) can be used to compute the load-settlement behavior of the shaft. Therefore, consideration of the method was in order. One must keep in mind that a much stronger and more rigid material (rock), with mechanical properties that may resemble those of the concrete, comes into the picture.

The formulation of the LTA is based on (a) the equation of vertical equilibrium in polar coordinates and (b) the assumption that the term related to the change of vertical stress with depth is negligible. Corresponding formulae are as follows:

(a) $T + r(\delta T/\delta z) + r(\delta \sigma/\delta z) = 0$, where

T = Lateral (or side) shear stress;

z = Vertical coordinate;

r = Radial coordinate;

σ = Vertical stress;

(b) $r(\delta \sigma/\delta z)$ term discarded.

It is easy to prove that if the (b) term is positive, then the computed shear stress (assuming no change

in σ) is larger than the "exact" value. The T-Z curves based on the stated assumption are, therefore, "conservative". The converse will occur if the (b) term is negative.

The lack of available information on the topic led the author to run some simple cases using a finite element code (linear elastic, axi-symmetric case), utilizing the mesh shown in Figure 4.1. The results, summarized in Figure 4.2, indicate the following:

A) For larger values of the ratio between the moduli of the concrete and the surrounding material, e.g. 1000 (case of a soil), the vertical stress tends to increase with depth, in general; only the zone close to the socket displays a different trend. The pattern seems to change if the modular ratio is of the order of 10 (case of a soft rock): only the zone at r=4m shows a more definite increment of vertical stress with depth.

B) The vertical stress gradient seems to decrease as the radial distance increases, being its magnitude larger in the zone close to the shaft. Therefore, it seemed appropriate to look at that region more closely. A finer mesh was used for the run, as depicted in Figure 4.3. The results (Figure 4.4) show that the vertical stress term is indeed smaller than the shear stress term.

The results just presented may serve as a guide for a more detailed study on this matter. In general, it appears that the basic assumption is fulfilled, i.e., the term related to the vertical stress is not important.

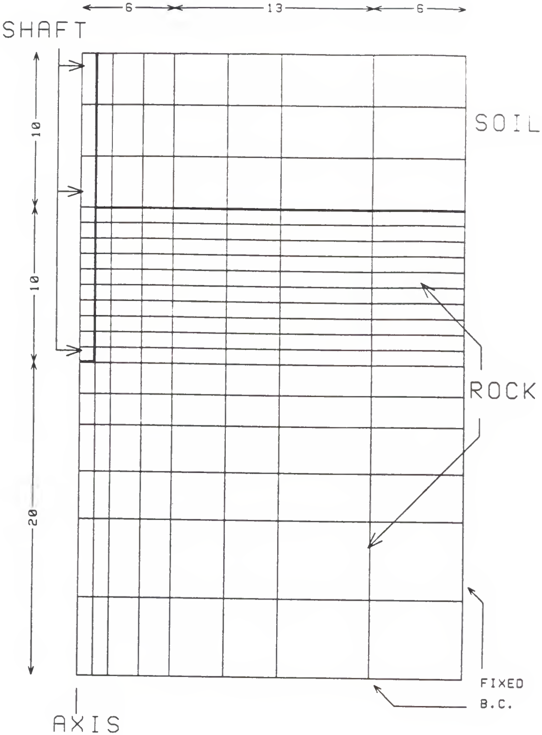
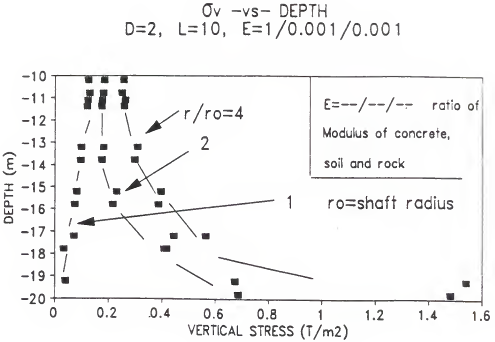


Figure 4.1. Vertical stress term. Finite Element mesh used for first runs.



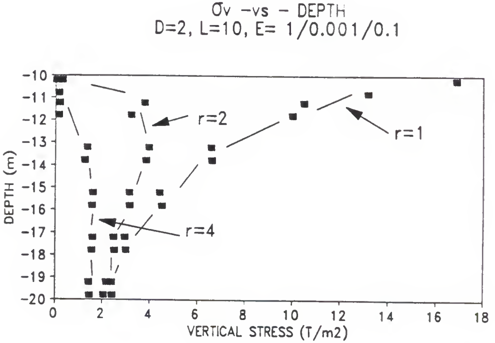


Figure 4.2. Change in vertical stress for sockets in soil and rock-type of situations.

Furthermore, the term tends to become even smaller for larger radial distances.

An attempt was also made to compare the results of the LTA and the Finite Element Method (FEM). The basic steps were as follows:

A) Run a case (FEM, linear elastic) that considered the full shaft (Figure 4.3) to compute the load-settlement at the top of the shaft.

B) Run independent FEM cases for each of the "anchors" to obtain the corresponding T-Z curves. It was decided to model three equally long anchors to cover the socket length; Figure 4.5 shows an example of the type of mesh used. The tip load-settlement behavior was found in an analogous way.

C) Input the computed T-Z curves into a LTA code (e.g. APILE2, written by Reese et al.) to obtain the butt load-settlement response.

The results can be abridged as follows:

METHOD	SETTLEMENT UNDER 500 T
FEM	0.00369 m
LAT	0.00376 m

The difference is about -2%, being the LAT approach "conservative".

Admittedly, the modeling covered a very narrow scenario (a single Length/Diameter and moduli ratio case) but may give some encouraging backup; the absence of

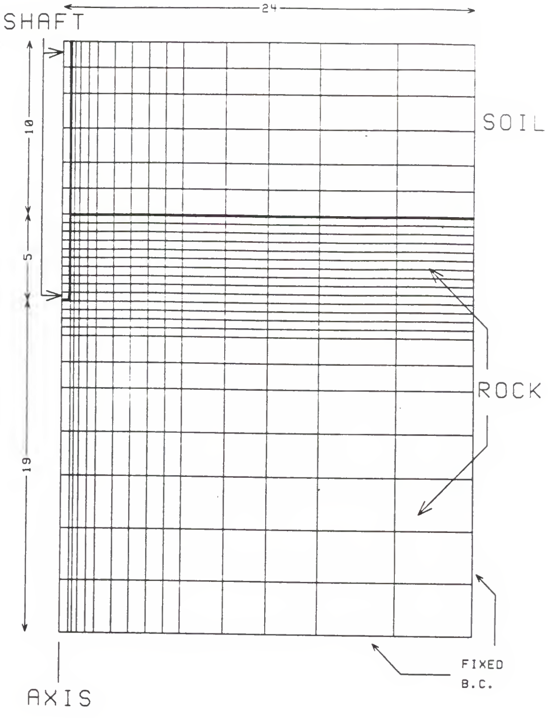


Figure 4.3. Finer Finite Element mesh used to
study the vertical stress term.

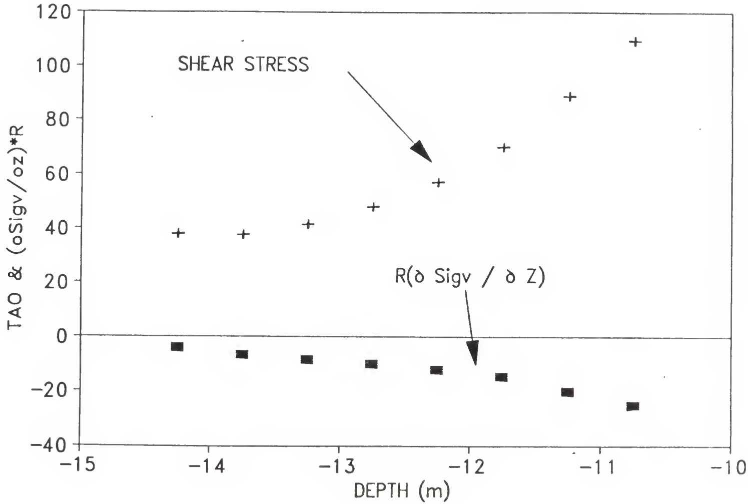


Figure 4.4. Term related to change in vertical stress.

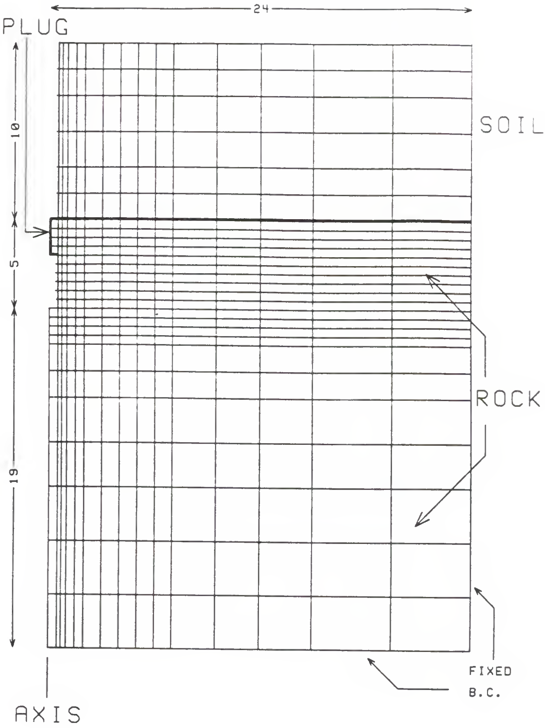


Figure 4.5. Example of finite element mesh used to model the plugs.

data on rock-shaft interface behavior may preclude the use of more sophisticated FEM modeling at this time. Nevertheless, some simple parametric analyses, considering various depths of embedment and shaft diameters could be performed as part of a future research.

In leaving this topic, one thing must be pointed out: the T-Z curves found via a load test on an instrumented shaft must yield the correct load-settlement when input into a LTA code. The curves involve all the terms, i.e., they include the one related to the vertical stress (the interaction between the different points of the shaft). One should also realize that errors associated with the measurements and the interpretation and reduction of the data may yield less-accurate T-Z curves.

Field Experimental Work

The field experimental work involved the following tasks:

A) Choosing the grout and the method to pour it.

B) Designing and building the system to set the anchors in the ground.

C) Designing and constructing the setup to measure load and displacement during a pullout test.

D) Choosing the testing sites and performing the exploration.

E) Designing and building a caliper to measure the diameter of the holes.

F) Installing the plug and performing the pullout test. This step included the collecting of data on some pullout tests done in the Miami Oolite.

Choosing the Grout and the Placement Method

Various home-made mixtures (based on some typical recipes) were tested, first. Three main requirements were defined, namely that the grout should flow easily through a small diameter pipe (1 to 1.5 inches in diameter), that it remain fluid for some time (at least 10 to 15 minutes), and that the shrinkage after setting be minimal. The results led the author to resort to some commercial products. Four grouts were tested: Setgrout, 713 grout, 928 grout, each made by Masterbuilders, and Instarock (a synthetic mix) produced by Metalcrete MFG Co.

The testing program, especially aimed at mixing, fluidity and strength characteristics, showed the following results:

A) A vigorous (preferably mechanical) mixing procedure must be used to get a uniform and fluid grout.

B) Shrinkage was minimal in all cases.

C) Unconfined compressive strengths in excess of 3000 psi could be expected after a 3-day curing period.

D) The modulus of elasticity was of the order of 280000 tsf. The values found for the Instarock were lower (smaller than 180000 tsf).

E) The grouts would flow through a pipe having a diameter between 1 and 1.5 inches.

F) Experiments run on the 928 grout showed that the mix remained fluid for at least 15 minutes.

Some considerations about the method to place the grout are as follows:

Tremie. Using a tremie has advantages and shortcomings:

A) Should one find that more grout is needed for filling larger-than-expected voids, it can be mixed and poured in immediately.

B) The flow of the grout may be turbulent, thus enhancing a cleansing effect on the wall of the hole. This effect may be specially true for anchors cast at larger depths.

C) The placement of a required volume of grout (to have a predefined anchor length, allowing for possible cavities) may be difficult to perform in a controlled fashion if done manually. Therefore, it could be difficult to ascertain the plug height at a given moment during the process. The use of a pump with volume measurement might help circumvent the problem, but also adds one more item to the required setup.

D) One must keep in mind that the size of the hole (about 6 inches) and the presence of the anchor's reinforcing cage leave little working space. The diameter of the tremie pipe may not be larger than say, 1.5 inches.

Such a reduced size may lead to "clogging" if the mixture is not uniform and fluid or if unexpected delays occur during the pouring procedure.

Using a container. A method by which the required grout volume (assuming, or measuring the hole diameter, and adopting a given anchor length) is prepared in a container and lowered to the required depth, was also considered. The scheme avoids shortcomings (C) and (D) cited above but does not yield the benefits expressed in (A) and (B).

The fact that no mud was to be used at the UF research sites led to the adoption of the container scheme. It was decided that the grout 928 (Masterbuilders) exhibited the required characteristics, namely a strength in excess of 3000 psi after a 3-day curing time and an elastic modulus similar to the ones reported for drilled shaft cases and about ten times larger than the value expected for the rocks.

Setup to Cast the Anchors

The setup (Figures 4.6 through 4.8) consisted of four main parts, namely:

A) A steel bottom plate and a nut welded to it. A 5-inch diameter plate size was chosen to get a little gap, should minor obstructions occur.

The assembly was used to pull the anchor from the bottom, via a Dywidag bar (1.375 inch diameter). Some

simple computations of bending moments and shear, considering a cantilever round plate, led to the choice of a 1.25 inch thick plate for the longer anchors (2 to 3 ft) and a 0.75 inch thick plate for the shorter ones (about 1 ft).

A seal was installed at the bottom of the plate to prevent the grout from escaping, i.e., flowing down past the bottom of the anchor; it consisted of a "lip" made of rubber, with a diameter a little larger than the diameter of the hole.

B) The reinforcing cage. Each anchor was reinforced with six #4 steel bars (60 ksi strength) covering the chosen anchor length. The lateral confinement was achieved by means of stirrups spaced at about 3 inches. The area of steel was computed using the ACI guidelines, assuming a range of possible maximum loads applied to the anchor.

C) The container. A plexiglass tube (5 inch inner diameter, 5.25 inch outer diameter) was used to contain the fluid grout. A metallic shoe was attached to one of the ends so the pipe could fit over the bottom plate tightly. A removable plastic lid was devised to cover the other end.

D) A metallic tripod was built using 12 ft galvanized steel pipes (1.5 inch diameter). The tripod was used to lower the cage and the plastic tube (containing the grout) into the ground, via a metallic cable and a manual winch attached to one of the legs.

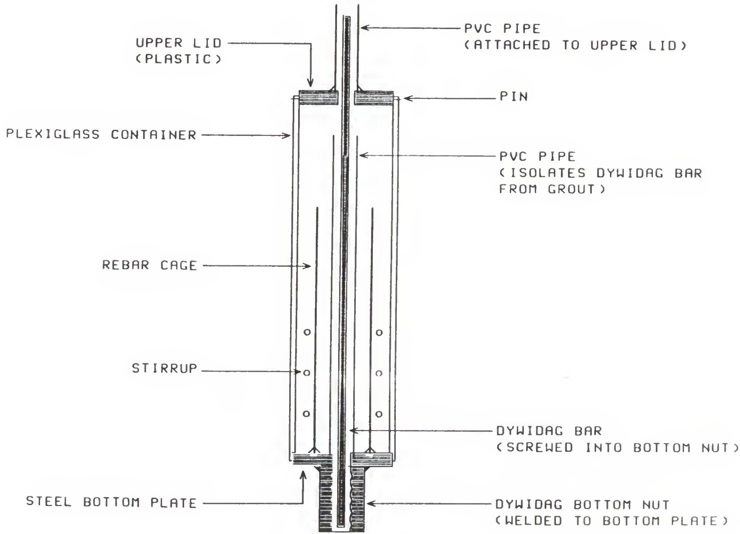


Figure 4.6. Setup to cast the anchors. UF design.

Figure 4.7. A view of the system used to cast the anchors appears on the left (note the anchor cage at the bottom left and the plastic container being held by Mr. Dobson). The grout was mixed with a drill, as shown in the picture at the right.

Figure 4.8. The grout was poured into the plastic container via a funnel, as shown in the left picture; note the upper lid and the attached PVC pipe. The plastic container (filled with grout) is ready to be lowered into the ground, as shown in the right picture

Setup to Apply and Measure Load and Displacement

The system to impose the pullout load (Figure 4.9) consisted of:

A) A Dywidag bar. The bar was threaded to the bottom nut (described before) so, when pulled from the surface, it imposed a pushing force at the bottom of the anchor. A plastic sleeve was used to isolate the bar from the grout so it could be unscrewed and recovered upon completion of each test.

B) A hydraulic 100-Ton hollow jack. An electronic transducer to measure the pressure was connected to the line that lifted the ram. The jack was then calibrated in a press, using a load cell to get the desired correlation, i.e., load - pressure transducer reading.

C) Reaction system . A 1.25-inch thick aluminum plate resting on the ground (actually, pads made with mortar were built to get a better working surface) was used for the first series of tests. A heavier setup consisting of a wooden grid and the metallic plate was utilized for the second series. The reasons for this change will be explained in the next paragraph.

D) The system to measure the movement of the anchor (Figure 4.10) consisted of a thin rope made of Kevlar (a high strength material with a high elastic modulus value, practically insensitive to temperature changes and creep effects under sustained load), a Linear Voltage Differential Transformer (a device that converts the movements of

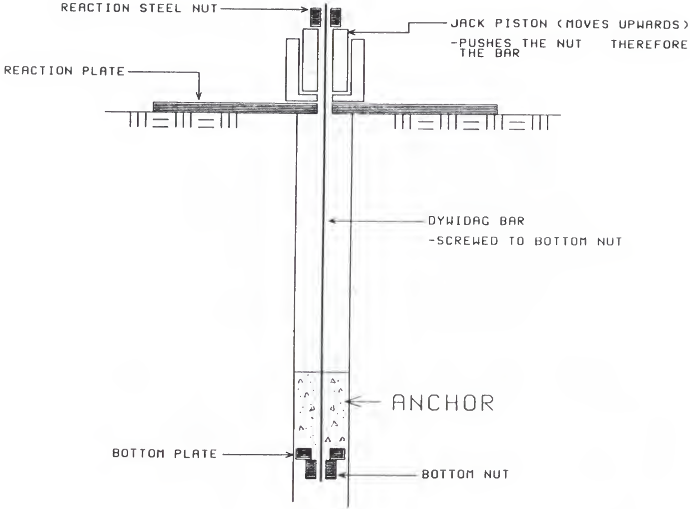


Figure 4.9. System used to impose the pullout load.

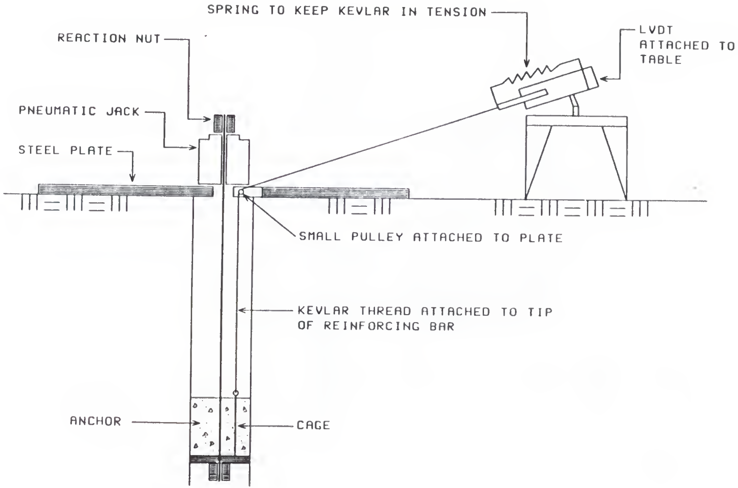


Figure 4.10. Setup used to measure the movement of the anchor. Initial design

a piston into a voltage signal), and a voltmeter. The thread was attached to the anchor's rebar cage on one end and to the piston of the LVDT on the other. The basic idea is that, with the thread under an almost constant stress, any movement of the anchor will translate into an equal displacement of the LVDT. This setup was used in the first series of tests, under the assumption that the effect of the settlement of the reaction plate (expected to be small) could be considered minimal. Some changes (Appendix A) were introduced in order to account for this effect as the tests proceeded. Eventually, a second scheme was devised to get a measuring setup independent of the loading system, as shown in Figure 4.11. The changes involved the reaction setup as well.

Site Selection and Exploration

The testing sites were chosen so that: (a) different rock formations were covered and (b) the T-Z behavior obtained via small scale tests (anchor pullout) and large scale (shaft load test), could be compared.

Three sites were selected as follows:

A) An abandoned quarry area located near Newberry (approximately 25 miles west of Gainesville).

B) A semi-active mining place located close to Gulf Hammock (about 40 miles southwest of Gainesville).

C) A construction site (North Dade Metro Parking Garage) located in Miami. Both pullout and a full size

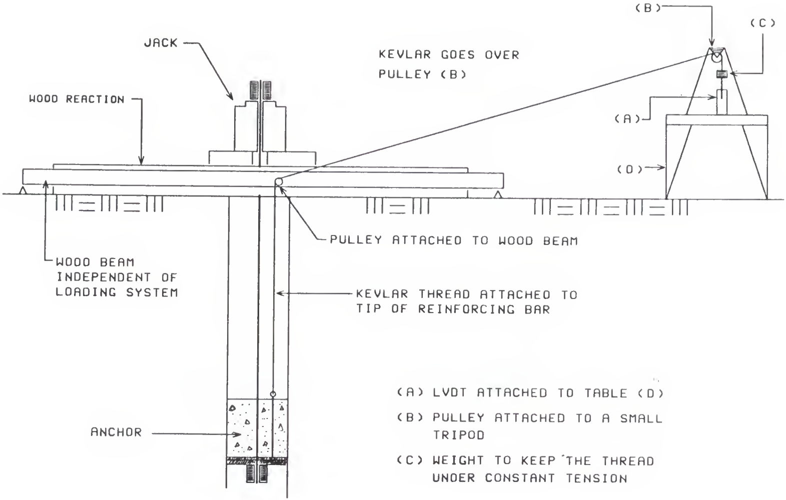
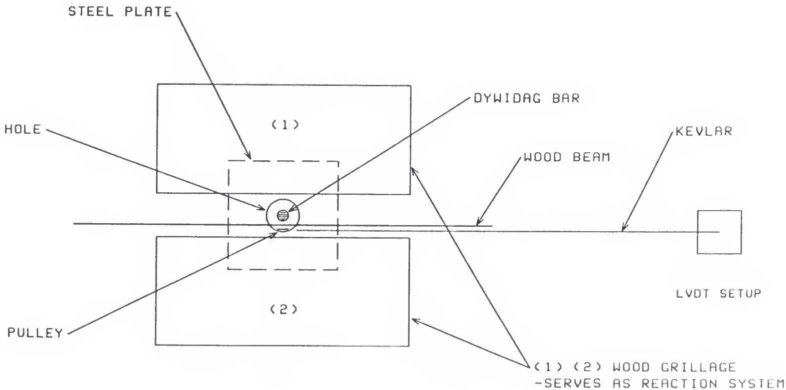


Figure 4.11. Setup used to measure the movement of the anchor. Final design (a) side view; (b) plan view.



PLAN VIEW

Figure 4.11--continued.

load test were performed at this site. The job was performed by Coastal Caissons, with Law Engineering as the geotechnical supervisor.

The first two sites were kindly provided by Florida Rock Industries Inc., who also performed the drilling.

Another area, the Brooksville quarry, owned by the above mentioned company, was also inspected. The presence of numerous cavities on the pit slopes as well as the apparent heterogeneity of the rock led the author to discard this site.

Newberry site

Drilling and surface reconnaissance. The general area is located about 25 miles west of Gainesville (Figure 4.12). An abandoned pit (about 100 ft by 100 ft), free from traffic, vibrations and any electric noise was selected.

The rocks that outcrop in the area belong to the Ocala Limestone (Miller, 1986), and are part of the so-called "Floridian aquifer". The formation was deposited about 60 million years ago; its age corresponds to the late Eocene.

Puri (1957) found the following types of rocks on the pit walls: (a) a shell coquina cemented in a granular matrix; (b) a granular limestone, and (c) a soft chalky limestone.

Figure 4.12. Location of research sites
(near Gainesville).

Eight holes were drilled for the present research, with depths that varied from about 7 ft to about 19 ft. The records can be seen in Appendix B. The recovery and the RQD were indeed low due to the friability of the rock and, at least in part, to the inexperience of the drilling crew. The 4 inch barrel does seem to be a common tool in mine exploration. The recovered cores agree with the rock types described before. The shelly and the chalky type of limestone prevailed, though. It must be pointed out that no overburden (or a very thin one) was present at the test site. The pullout reaction system rested on bare rock in practically all the cases. No water table was found in any of the holes. A definite loss in drilling fluid (water) was recorded at 15 ft in two of the holes. No special losses were detected above that level.

The surface reconnaissance disclosed the following points:

A) The rock mass seems to be uniform. A few distinct seams (estimated to be 1 ft thick) of weaker and more erodible material seem to be interbedded within the rock mass. They may explain, in part, the presence of very weathered stretches and/or the sudden loss in drilling fluid encountered during the drilling work.

B) No evidence of well defined vertical joint sets was detected. An exception occurred in the testing area. A vertical crack with clear signs of dissolution was present on one of the pit walls. The crack runs across the site

though the dissolution is not so marked, at least on the surface.

C) The strata seem to be horizontal. Any minor dip is irrelevant considering the size of the testing area.

Seismic exploration. A number of cross-hole tests and a single down-hole test were performed. The main purposes of the tests were:

A) To get some information on the rock mass body (P) and shear (S) wave velocities.

B) To compare the field values with those found via nondestructive (seismic) testing in the lab. This comparison may yield some information about the relationship between mechanical properties of the mass and the "intact" cores.

The tests were performed by Professor Richard Woods with the collaboration of the author and co-workers. The plan was accomplished without special difficulties. Some minor changes had to be introduced in the setup due to the fact that the original system to hold the geophones was designed for holes with a smaller diameter. The basic steps for a typical test can be abridged as follows:

A) The testing holes were chosen, first. The horizontal distances between them were carefully measured with a measuring tape.

B) The hammer (source) and the geophones were set at the chosen depth. All the instruments were located at the same level.

C) The hammering was pursued until several signals were analyzed. A two channel, 5 microsecond discretization time Nicolete Oscilloscope was used for this research.

D) The procedure was repeated for different depths as well as for different hole arrangements.

The down-the-hole technique was performed in a single hole; the hammer and the geophones were set at the chosen depths, and the hammering repeated until several signals were analyzed. The procedure was repeated, setting the instruments at different elevations each time, to test a different stretches each time. A summary of the results is presented in Appendix B.

Gulf Hammock site

The site is located approximately 40 miles south-west of Gainesville (Figure 4.12). The research site was located in an area where the mining operations were barely noticeable at the time.

The rock that occurs in the area corresponds to the Avon Park Formation, according to Schmidt et al. (1979). This formation is the oldest that outcrops in the state (late Middle Eocene) and consists essentially of a "...tan to brown, thin bedded, laminated, finely crystallized dolomite. The Avon Park varies from very porous and soft to dense and well indurated. Layers of fine silt-sized dolomite occur..." (Schmidt et al., 1979, p.6).

Two holes were drilled to depths of about 15 ft. The drilling records are shown in Appendix B. The rocks

found agreement with the descriptions just presented. The recovery and the RQD were high (in general larger than 80%), and quite a few good cores were recovered. The core barrel was successfully used this time.

It must be pointed out that no overburden (or a very thin one) was present at the test site. The pullout reaction system rested on bare rock in all the cases.

The water table was located 5 ft below the ground surface. No losses of drilling fluid (water) were recorded at any time in either hole.

The abandoned pits are currently flooded so it was not possible to get any information on joint patterns or other features. The similitude displayed by the two borings (similar rocks at similar depths) show that the strata are horizontal, at least on a local basis.

Miami site

The area is located in Dade County, at the intersection of US 1 and Kendall Avenue. A large parking garage is being built as part of the Metropolitan Dade County's Rapid Transit system. The building will be supported on shafts drilled into the Miami Limestone and the Fort Thompson Formations.

The geology of the area has been reported by Prieto-Portar 1981; a summary will be presented, as follows:

Miami limestone (Miami Oolite). The Miami limestone "...is a soft-to-medium porous, sometimes sandy, fossiliferous, pelletal oolitic grainstone...sand-sized calcium

carbonates were precipitated around shells or sand grains called oolites. . . The deposit was formed during an interglacial period 120,000 years ago." (Prieto-Portar, 1981, p.364).

The thickness of this formation is about 21 ft along the transit corridor (Prieto-Portar, 1981).

Fort Thompson. "The lithology of this formation is highly variable, and is composed primarily of sandy limestones interbedded with fresh-brackish water limestone, coralline limestones, unfossiliferous fine quartz sands, and molluscan quartz sandstones." (Prieto-Portar, 1981, p. 368).

There are two distinct strata (Prieto-Portar, 1981) on top of the formation (i.e., just below the Miami Oolite): a loose fine grained unfossiliferous quartz sand (3 to 26 ft thick), overlying a compactly cemented sandy limestone (up to 23 ft thick, with high carbonate content).

The results obtained by the above mentioned researcher (Figure 13 in Prieto-Portar, 1981) indicate that the degree of variability of the Fort Thompson Formation is apparently larger.

Three holes were drilled at the site by Coastal Caissons under the supervision of Law Engineering. Copies of the records are included in Appendix B.

Design and Construction of a Caliper

As the casting of the anchors at the Newberry site progressed, the author realized that the holes were somewhat irregular. The measured height of the plugs was sometimes noticeably smaller than expected, indicating that either the bottom seal was not effective, the hole diameter was irregular, or both effects were present.

Therefore, the author decided to develop a device to log the diameter of the holes. Although commercial calipers are available, they are designed to be used in full size shaft holes (of the order of 30 inches or larger).

A simple mechanism was designed and built at the UF structures-geotech workshop. A hinged truss, attached to a pipe on one end and free to slide on the other, is used to convert horizontal movements (due to change in diameter) into vertical displacements. The latter are then measured by means of a LVDT which is connected to a chart recorder. The setup includes a hydraulic system to either collapse the caliper (should the device get stuck in a cavern), or to keep it open under a gentle pressure. Figures 4.13 and 4.14 may give a better idea of the system.

The caliper is inserted in the hole and a pipe, or a cable, is used to move it along the hole. Using a pipe allows one to control the traveling path, i.e., the

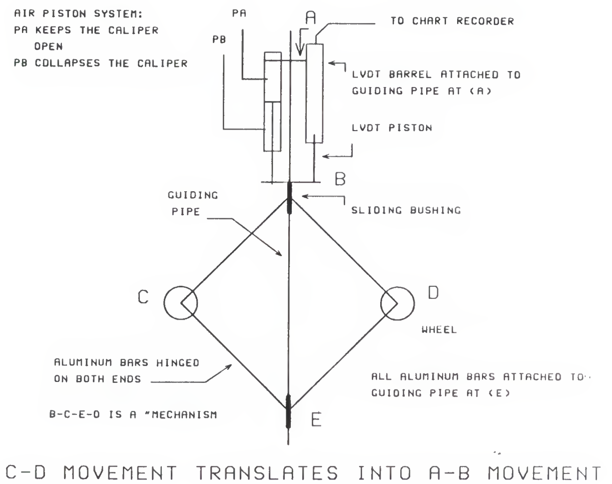


Figure 4.13. Sketch of UF caliper to monitor the size of the hole.

Figure 4.14. View of UF system (caliper and chart recorder) to monitor the size of the hole.

orientation of the device. however, this system is a little bulky and cumbersome. Using a cable (the device is pulled from the surface) makes the test simpler, but the orientation can not be assessed. Several travels can be made to get an idea about the profile of the borehole.

One must keep in mind that the detection of local cavities or vertical cracks may or may not be accomplished. A more sophisticated design (several wheels and perhaps several LVDTs) would be required to get a better "coverage".

The device was used in the last stages of the testing program at Newberry. It was utilized in Miami, with partial success, and at Gulf Hammock.

Anchor Installation

UF Research sites (Newberry and Gulf Hammock).

The installation of each plug consisted of the following steps:

A) The tripod was centered over the hole and the Dywidag bar set hanging from it.

B) The upper lid (see Setup to cast the anchors) and the plastic container were slid along the Dywidag bar, in that sequence. The Kevlar thread was inserted through a little hole located on the upper lid.

C) The anchor was screwed to the bar. The Kevlar thread was attached to the tip of one of the reinforcing steel bars.

D) The plastic container was slid until the metallic shoe was set tight on the metallic bottom plate; the upper lid was held at a higher elevation by means of pliers that grasped the Dywidag bar.

E) The required amount of grout was mixed in a plastic bucket. A plaster paddle attached to a 1/2 inch hand drill was used to ensure a fluid and uniform material. A mixing time of not less than 7 minutes was adopted, according to the suggestions of the manufacturer. Iced water was used to extend the setting time.

F) The upper lid was then lowered and attached to the plastic container using four pins; a piece of duck tape was wrapped around the pins to prevent them from falling.

G) The whole arrangement was carefully and steadily lowered to the test depth, using the winch.

H) The plastic container was pulled from the surface by means of ropes attached to the upper lid (remember that the units were attached to each other). The grout could flow and form the anchor.

I) Samples of the grout were recovered for unconfined compression tests; 3-inch by 6-inch cylinders were used.

J) The grout was allowed to harden for at least 4 days. The elevation of the top of the plug was then measured using either a measuring tape or a steel bar. The reaction system as well as the jack could then be set.

The loading test procedure can be summarized as follows:

A) The electronic setup (LVDT, pressure transducer and corresponding power supplies) were hooked up. A converter was used to withdraw the power from a car battery.

B) The displacement measuring system (LVDT attached to a carpenter's table) was installed about 10 ft away from the reaction system. The free end of the kevlar rope was attached to the tip of the LVDT.

C) The mechanical pump was connected to the jack. This operation could be performed as (B) and (C) were executed.

D) Initial readings of the displacement measuring and the load measuring systems were taken.

E) The estimated maximum load was split into 10 increments, at least. Each load increment was sustained for 4 minutes, recording the LVDT readings at 0, 2 and 4 minutes. The procedure was repeated until the load reached a peak (usually accompanied by large displacements). Some measurements were also taken in the post-peak region.

F) The Dywidag bar was unscrewed once the test was completed.

An attempt was made to recover a plug at the Newberry site. The reaction system tilted a little during the process and, as the jack kept pulling (force of the order of 60 Ton), it bent the Dywidag bar. The attempt was

quickly abandoned to avoid any further damages to the setup.

Five pullout tests were performed at each site. Pertinent information can be seen in Tables 4.1 and 4.2. as well as in Appendix C.

TABLE 4.1 Newberry Site Pullout Test Data

Test #	Depth(ft)	Date Installed	Date Tested
1	4 - 5	May11/90	May 16/90
2	6 - 6.5	" 17	" 22
3	11.7 - 14.5	June11	June18
4	8 - 9	" 21	" 25
5	4.6 - 7	July6	July13

TABLE 4.2 Gulf Hammock Site Pullout Test Data

Test #	Depth(ft)	Date Installed	Date Tested
6	11.8 - 13	Oct19/90	Oct24/90
7	7.7 - 9.7	" 29	Nov2
8	13.4 - 14.2	Nov5	" 9
9	8.8 - 10.8	" 19	"26
10	5.3 - 7.3	" 28	Dec5

<u>Miami site</u>

The sequence of the installation of the anchors can be summarized as follows:

Three holes were drilled to depths of 55 ft, 60 ft and 54 ft, respectively, using a 4-inch core barrel. All were drilled to almost the same depth in an attempt to better define the stratigraphy of the testing area. The drilling proceeded with some difficulties, due to cave-ins in the sandy stretches. The continuous addition of

drilling mud did not seem to control the problem. It should be pointed out that no special controls were imposed, either on the density or the viscosity of the mud.

Upon completion of the holes, a decision was made by Coastal Caissons and Law Engineering concerning the depths and the sequence in which the anchors should be installed and tested. The deepest anchor (named PT3, set at about 48 ft) was cast first in the hole drilled most recently; the second (PT2, at 55 ft) was set in the second hole, one day later, and the third (PT1, at 30 ft) was installed in the hole that had been drilled first, one day later. The diameter of each hole was enlarged to a depth close to the selected anchor level. This operation was meant to facilitate the recovery of the plugs after the tests.

Except for the case of PT1, the anchors were installed without filling the hole stretch below them with gravel or grout in a controlled way. Apparently, some debris and the cuttings originated by the overdrilling (see previous paragraph) filled the hole to the required depth.

The cleansing procedure was accomplished by flushing water for about half an hour in each hole.

The installation of each plug was accomplished according to the following procedure:

A) A tremie pipe (PVC, 1.5 inch diameter) was set in the hole. A "traveling plug", i.e., a piece of cloth, was used to avoid the inflow of water and mud into the

pipe. The grout (Setgrout, manufactured by Masterbuilders) was mixed by hand using a trowel and was put in the tremie, gradually. As explained before, water and mud remained below the selected anchor depth, so grout had to be poured continuously to fill the hole up to that level. The continuous addition of grout made any guess about its current elevation in the hole, difficult. Several techniques were attempted to assess such level: the changes in the water table position were monitored; a weight, attached to a measuring tape was used as a probe to "feel" the change in density; a mud sampler was lowered in the hole to ascertain whether the grout had reached the desired elevation or not. The last technique seemed to be successful.

B) The anchor reinforcement was then lowered to the selected depth. The procedure looked simple at first sight but turned out to be somewhat complicated and time consuming, at least for PT3. Ramos (Law Engineering supervising engineer), noted that the insertion could have dragged some debris and mud that could have contaminated the grout.

The author's attempts to use the caliper were somewhat unsuccessful. The worries about the stability of the holes coupled to contractual time constraints did not leave room for a thorough utilization of the device.

The Kevlar thread could be installed in plugs PT1 and PT3 just before the cages were lowered into the holes;

unfortunately, this last one was twisted during the installation so the thread got somewhat entangled. A different procedure was used for PT2. The Kevlar was attached to the tip of a steel bar, which in turn was inserted in the hole until it touched the top of the anchor.

The pullout test procedure was quite similar to the one described before. Time steps of 5 minutes were used, though.

Table 4.3 shows relevant data concerning the test program.

TABLE 4.3 Miami Site Pullout Test Data

Test #	Depth(ft)	Date Installed	Date Tested
1	28.8 - 32	Aug28/90	Sep3/90
2	53.8 - 57	" 27	" 2
3	46.2 - 49.5	" 26	" 1

Each plug was recovered after the pullout test so an estimate of the effective rock-grout shear area could be made. Special attention was given to the presence of grout that seemed to be contaminated with mud or loose pieces of rock.

Collection of Pullout Data (Miami Oolite Formation)

Data on six pullout tests performed for the Miami Metromover project were graciously provided by Law Engineering (Miami Offices). Some rock cores recovered near

the pullout sites were still available, so non-destructive (lab seismic) tests were performed on them in Miami.

Table 4.4 shows information about the tests.

TABLE 4.4 Miami Oolite Pullout Test Data

Test #	Depth(ft)	Date Installed	Date Tested
1	8.5 - 14.9	----	----
2	8.4 - 14.8	Apr25/90	May14/90
3	9.4 - 14.6	" 24	" 11
4	10 - 14.8	" 24	" 14
5	9.8 - 14.9	" 24	" 11
6	8 - 12.5	" 23	" 14

Laboratory Experimental Program

The lab testing program involved the following steps:

A) A careful trimming of all the rock cores that seemed amenable for destructive and/or nondestructive testing. Any piece 2 inch long or larger was taken into account.

B) Designing a simple setup to perform the non destructive seismic tests.

C) Performing the lab seismic tests.

D) Performing the destructive tests, namely unconfined compression and split tensile.

Seismic Tests

A program of nondestructive seismic testing was performed on most of the available rock cores. A summary of the techniques used can be presented as follows:

A) A device brought to UF by Professor R. Woods was used initially; it consisted of body (P) and shear (S) wave geophones (connected to an oscilloscope) that were pressed to the ends of a given core. A high frequency generator was used to set the pulses into the sample. The power of the generator did not seem to be great enough to provide strong signals, thus making the interpretation of the P and S wave arrivals difficult. The problem seemed to be related to the porous character of the cores, i.e., the energy was being dissipated inside them.

B) A second technique was attempted. The core was hit on one end with a little hammer (the triggering accelerometer was attached to it) and the arriving signal picked up with another accelerometer, glued at the other end. The accelerometers were connected to a digital Nicolete 4562 oscilloscope. This device allows one to define the beginning of the excitation (triggering signal) and the moment it reaches the other end of the rock piece. A digital readout indicates the difference in time between the signals. The wave speed can be found by dividing the travel length (measured from the point of impact to the second accelerometer) by the travel time.

C) If one considers that the shaft side shear is related to the shear rigidity of the rock, it appears that the testing program should focus on the determination of shear (S) wave speed. This led to the design and assembly of an apparatus that imparted an excitation rich in shear.

A very simple torsion-type of device (Figures 4.15 and 4.16) was considered appropriate for this purpose.

Shear wave tests were conducted on practically all the cores recovered at Newberry. Although the geophones were aligned to detect a shear wave, P-waves were detected in most cases. The reversed-polarity scheme was attempted but, still, the computed S-wave speeds were too high in some cases. Furthermore, some concern arose in connection with the amplitude of the excited waves. Some computations indicated that frequencies over 25000 Hz were required to have a wavelength no larger than half the travel length, while the inspection of typical records showed that this requirement was not being fulfilled.

D) Another approach was attempted, using a more powerful high frequency source to generate the waves. A CSI, type RBT 2A concrete tester unit was utilized for this purpose. The device had an excitation unit that could impart P or S waves depending on how it was oriented on the sample. The accelerometers had to be aligned accordingly, as depicted in Figure 4.17.

Most of the Newberry cores, the ones collected at Gulf Hammock, and the samples provided by Law Engineering in Miami were tested using the scheme described above. P and S-wave determinations were performed on each core. The determination of the S-wave arrival was somewhat easier, since the P-wave time was already known (this type

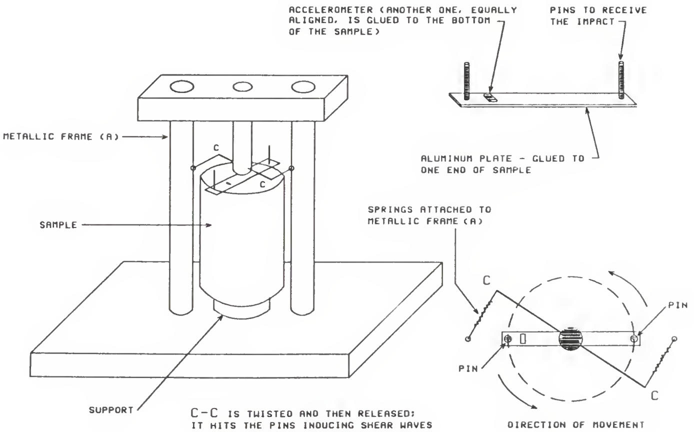
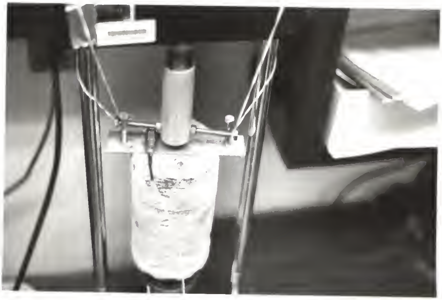


Figure 4.15. System designed to induce shear wave pulses.



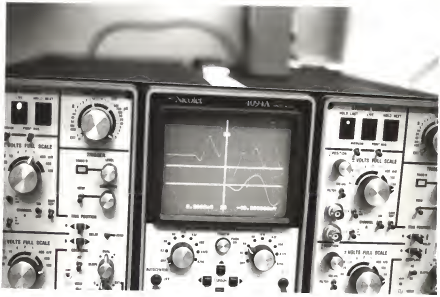


Figure 4.16. Partial view of the setup used for P and S-wave determination. One of the accelerometers and the oscilloscope are shown.

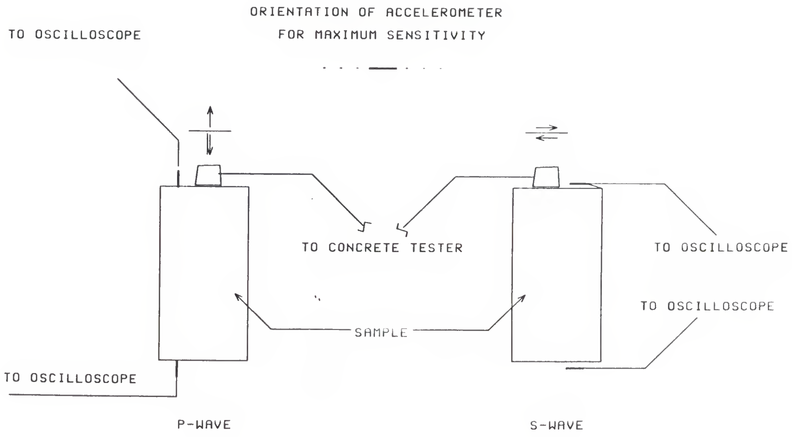


Figure 4.17. Orientation of accelerometers for P-wave and S-wave testing.

of test was always performed first). Impact-type tests (vertical and torsional excitation) were also repeated to see the effect of wave frequency and wave length.

Destructive Tests

Unconfined compression and split tensile tests were performed on selected samples. Cores obtained at Newberry and Gulf Hammock were tested at the University of Florida. Law Engineering handled the cores from Miami Dadė Station and Miami Metromover. The basic procedure agreed with the ASTM standards (rate of load application, seating procedure prior to stress-strain determination). Most of the specimens were capped with sulfur. A mechanical device (used in concrete testing) was utilized to measure the deformation of the core, in order to find the stress-strain curve (compression tests) of some of the samples.

CHAPTER 5
RESULTS AND DISCUSSION

This chapter summarizes the field and lab findings for each of the studied sites. Some new information related to the following topics will be presented:

A) The anchor installation procedure, including preliminary steps like the measurement of the diameter of the hole and the assessment of the dimensions of the plug.

B) The technique to measure force and displacements throughout the pullout test.

C) The T-Z behavior, i.e., the shape and slope of the curves, the displacement required to reach the peak shear stress value, and the degree of post-peak softening.

D) The non-destructive (seismic) testing, including comparisons between high frequency and impact tests, comments on compression (P) and shear (S) wave speed determinations, and comparisons between field and lab test results (Newberry case).

E) The relationships between results from lab seismic (very low strain) and destructive (strength) tests.

F) The verification of published data on relationships between drilled shaft maximum side shear stress and

rock strength (found via unconfined compressive and/or split tensile tests).

G) Some points of departure for correlations between lab seismic and field pullout test results.

Anchor Installation. Hole Inspection

Two methods were used to cast the anchors in the ground, namely: (a) the tremieing technique (Miami Site, Miami Oolite tests) and (b) a scheme by which the anchor is first formed on the surface (using a container to hold the liquid grout) and is lowered to the chosen depth afterwards (Newberry and Gulf Hammock sites). Both techniques were successful, though each displayed some limitations.

Tremie (Miami Site)

Assessing the level of the grout in the hole may be difficult, especially if the stretch underneath the anchor has not been previously filled with material in a controlled fashion. The quality of the grout may be affected because the flow from the tremie is not fully deflected upwards (case of a solid bottom), thus favoring a cleansing process. Instead, part of it may go downwards (case of a muddy bottom), which creates a complex flow situation.

The scheme used at the Dade Station, i.e., pouring the grout and lowering the reinforcing cage afterwards, looked simple. However, the cage could get covered with

mud during the process and contaminate the grout. A method in which the cage is set first and the grout poured right after, seems more appropriate.

The occurrence of the two factors above mentioned may help explain the weak (contaminated spots) found in all the recovered plugs at the Miami site.

It should be pointed out that anchors PT1 and PT2 were installed in holes that remained filled with stagnant drilling mud for about two days.

Container Method

The success of the technique depends in part on the proper functioning of the bottom seal (Chapter 4). The experience collected at Newberry showed that the seal may be ineffective, specially if the hole is irregular; this indicates that the hole must be filled to the chosen elevation before proceeding with the installation of the anchor.

It should be pointed out that the setup was used to cast anchors at relatively shallow depths (15 ft at the most). Some modifications will have to be introduced to use it at greater depths, of the order of 50 ft. The changes would be to the system used in lowering the container and to the system used in pulling it, thus allowing the grout to escape and form the anchor; a larger tripod and a sturdier system with steel cables would be required, respectively.

Assessing the Plug Dimensions. Use of a Caliper

Any information about how the size of the hole changes with depth is useful, for these reasons: (a) it allows one to choose regular stretches to set the anchors; (b) it provides clues that may help guide the pullout interpretation procedure (presence and extent of cavities or weaker spots, for example), and (c) it may aid in making a decision on whether plug recovery is necessary or not, from the stand point of geometry.

The first four anchors at Newberry were constructed under the assumption that the diameters of the holes were on the order of 6 inches (according to the inspection of the upper part of the holes). As the program proceeded, it turned out that the expected anchor lengths were in most cases much smaller than anticipated. Some reasonable explanations were that the bottom seals were ineffective, or that the holes were irregular. A caliper (described in Chapter 4) was built and used. The chart records (Appendix B) show that the holes had larger diameters than expected at various depths. Only one anchor (#5, the last one) was set after the size of the hole had been recorded. The use of the device at the Miami site was quite limited, as explained before. Two records gathered in pullout #3 hole showed that the bottom 1.5 ft of the chosen stretch had a diameter of about 5.5 to 6 inches, while the upper 1.5 ft seemed to have a larger diameter, close to 7 inches. The

caliper was used repeatedly at Gulf Hammock. The records (Appendix b) indicated the presence of uniform as well as irregular stretches.

An assessment of the average diameter of the anchors at Newberry and Gulf Hammock was done as follows:

A) The elevation of the <u>top</u> of the anchor was measured, once the grout had set. A bar was inserted several times until the "hard" grout had been detected at various locations. The elevation of the <u>bottom</u> of the anchor was known by subtracting the protruding length of the Dywidag bar from the total length. The difference in elevations (top to bottom) yielded the anchor length.

B) An average diameter was computed by dividing the volume of grout by the anchor length. This average diameter and the anchor length were used to compute the side area, i.e., the one on which the shear resistance occurs.

A comparison between the diameter found by this scheme just described and the one recorded with the caliper was made. The findings can be summarized as follows:

SITE	COMPUTED DIAMETER	CALIPER DIAMETER
Newberry, plug 5	6.5 in	6.0
Gulf Hammock	5.5-5.8	5.5-5.8

The disagreement in the first case may be due to the fact that some grout could have escaped through the bottom seal. The volume of grout used for the computations was larger than the true one.

As shown in Appendix B, Gulf Hammock Hole #1 seemed to be somewhat irregular from 8 to 10 ft. The fact that the amount of grout used in plugs #1 and #2 yielded anchor lengths close to the nominal values, may lead one to think that the caliper disclosed a local cavity. It appears that the device traveled almost the same path every time it was inserted in the hole.

All three anchors (Figure 5.1) were recovered at the Miami site. The effective area was estimated visually. Those patches where some contamination was suspected and those where the grout had been peeled off were discarded. It may be interesting to notice that the diameter of PT3 (bottom 1.5 ft) was about 5.7 to 6 inches, i.e., similar to what the caliper recorded. The upper 1.5 ft (where the caliper disclosed an abrupt change in diameter) showed a grout that was peeled off.

An inspection of the anchors after the test gives valuable information about their height and diameter and, most important, it allows one to determine if the grout could have been contaminated. However, the continuous drag may destroy part of the plug, thus leading one to assess a smaller-than-real side area, and therefore to compute a higher (unconservative) shear stress. The assessment of an "effective" area where the anchor sheared the rock is to some extent subjective. For example, some patches may be discarded because the grout was peeled off but it is feasible that pieces of the anchor fell in the hole as the

former was extracted. One point which must be kept in mind is that only one test can be performed in a given hole if the recovery is mandatory.

Figure 5.1. View of the anchors recovered at the Miami site. PT1 (left one) failed through the grout. PT2 (center) and PT3 (right) show symptoms of contaminated grout, especially at the top end. The areas where the grout was peeled off were neglected for computing the side shear.

Load and Displacement Measuring Systems

The pressure transducer used to monitor the applied load worked quite well throughout the testing program. Some problems related to temperature effects were quickly handled by covering the device with a shade. The system was calibrated at the beginning and at the end of the program, and the linear relationships between pressure and load were found to be unchanged.

The scheme used to monitor the movement of the anchor proved to be very simple to handle. The first design was somewhat more cumbersome, for the settlement of the reaction plate also had to be recorded. The final design was more expeditious and accurate, according to some calibrations run in the lab. The technique has two distinct advantages: (a) temperature changes should not affect the results and (b) the monitoring of quick movements associated with a sudden type of failure, as well as the post-peak behavior, can be handled better and more comfortably.

The movement of an anchor was found in a different way at the Miami Site. The stretching of the Dywidag bar (computed using the load and the rigidity of the bar) was subtracted from the total movement of the tip of the bar (measured with dial gages attached to a reference beam). Such a scheme may exhibit two disadvantages, namely: (a) the effect of temperature changes and (b) the possible

effect of minute deformations and slippage of the Dywidag couplings.

The above described techniques could be compared reliably in one case, i.e., pullout T1 (Miami Site). Both gave similar results (0.669 inches UF vs. 0.633 inches Law Engineering-Coastal) at the moment of failure. Somewhat different values were computed on the post-peak region (1.02 inches UF vs. 1.36 inches Law Engineering-Coastal). It may have happened that the dial gage needles moved quickly, thus making it difficult to keep track of the readings.

T-Z Behavior

Newberry Site

The T-Z curves obtained at the Newberry Site can be seen in Figures 5.2 through 5.5. The first test was considered unreliable and was discarded. The behavior can be summarized as follows:

A) The curves showed various shapes. No apparent trend to a specific fit, e.g., a hyperbolic, could be defined.

B) The maximum side shear varied between 25 and 30 tsf, for anchor lengths between 0.6 ft and 2.8 ft. The variability of the rock, coupled to the irregularity

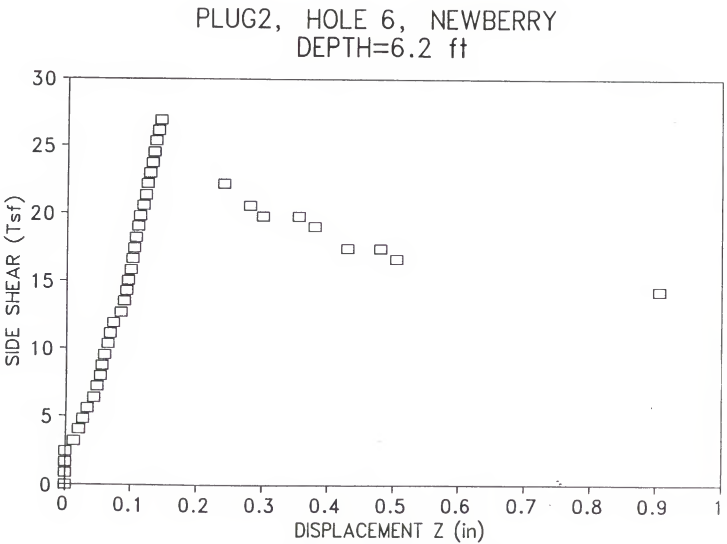


Figure 5.2. T-Z curve of plug #2, Newberry Site.

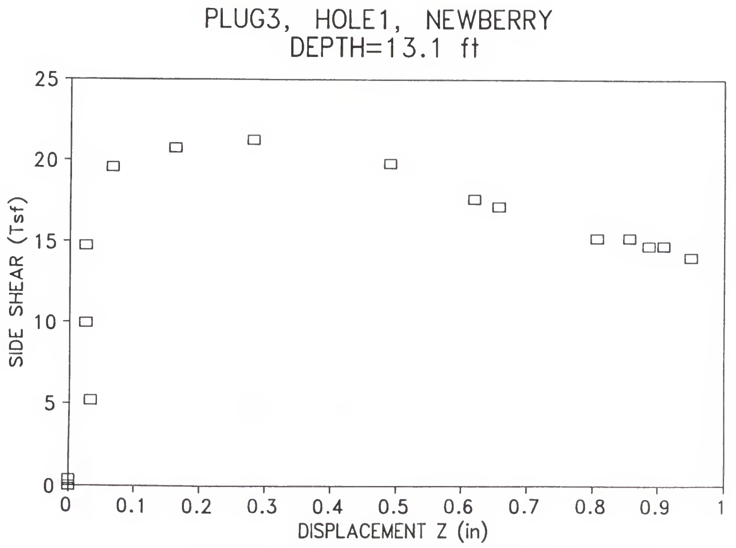


Figure 5.3. T-Z curve of plug #3, Newberry Site.

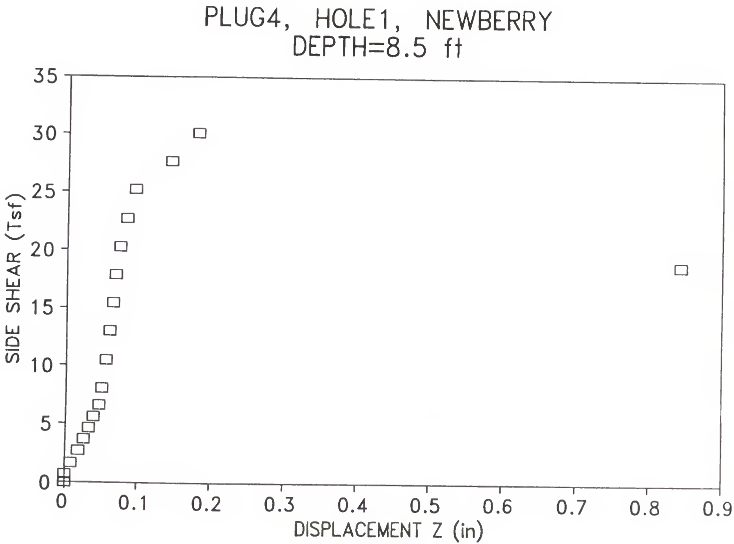


Figure 5.4. T-Z curve of plug #4, Newberry Site.

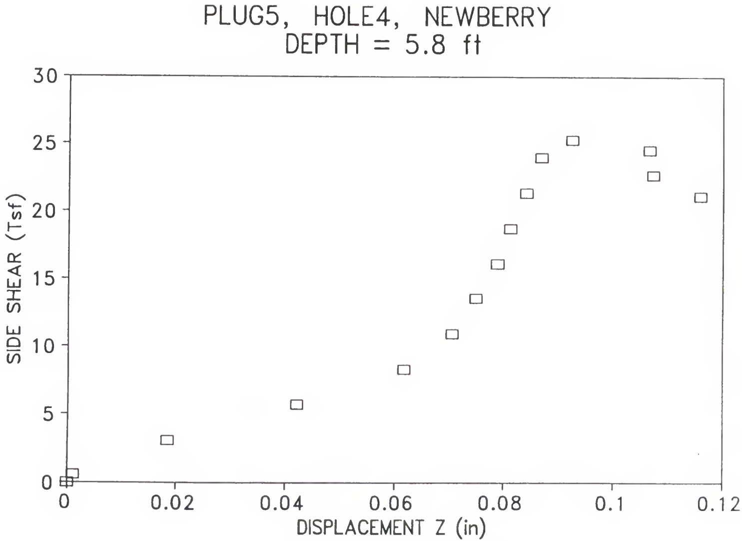


Figure 5.5. T-Z curve of plug #5, Newberry Site.

of the holes, makes the study of the effect of the anchor length/anchor diameter ratio on the maximum side shear, difficult. The influence of this parameter may not be very prominent, as judged from the observation of the range of computed shear stresses (25 to 30 tsf, i.e., 20% difference), the uniaxial compressive strength of the rock (110 to 164 tsf) and the wide L/D ratios tested (from 1 to 5.6).

C) The displacement required to mobilize the maximum side shear varied from 0.1 inches to 0.3 inches. Four of the five tests yielded values lower than 0.2 inches.

D) The degree of softening, i.e., the ratio between the lowest post-peak value and the maximum shear, varied from 0.5 to 0.8; the smallest value corresponds to a short anchor (0.6 ft long). This may indicate that the anchor-rock interface is more on the "rough side" according to the results presented by Rowe and Pells (1980). Based on a visual inspection, the <u>roughness of the cores</u> could be classified as R2 to R3 in Pells et al. (1980) system. Another point to take into account is that the plugs were set at relatively shallow depths (less than 15 ft). Therefore, the expected geostatic stresses are low, thus favoring a brittle type of behavior (Rowe and Armitage, 1987).

E) The secant slope corresponding to the maximum shear lies between 75 and 275 tsf/inch, with most of the values larger than 150 tsf/inch.

Figure 5.6 shows a semi-dimensionless summary of the T-Z curves. The shear stress was divided by the

maximum side shear, without modifying the abscissae. It must be remembered that the scale effect was assumed to be minimal at the onset, so the anchor displacement/anchor diameter quotient was not considered to be useful or necessary.

Gulf Hammock Site

Figures 5.7 through 5.11 display the T-Z curves corresponding to the Gulf Hammock Site. The observed behavior can be summarized as follows:

A) The curves show a steep initial stretch, with a kink where the side shear reaches a maximum value. Again, no simple hyperbolic fit could be found.

B) Maximum side shear values from 13.6 to 19.5 tsf were computed for the more porous dolomite, the higher value corresponds to a sounder rock. The L/D effect does not seem to be very important. This ratio was varied from 1.5 to 4, with corresponding shear stresses that differed by 20% to 25%. Test #5, run in a much stronger rock (finely crystallized dolomite), yielded a maximum shear stress of 50 tsf.

C) The displacement required to mobilize the maximum side shear varied from 0.08 inches to 0.11 inches.

D) The degree of softening varied from 0.6 to 0.8; the smallest value corresponds to a shorter (about 1 ft long) anchor. The comments on the roughness of the anchor

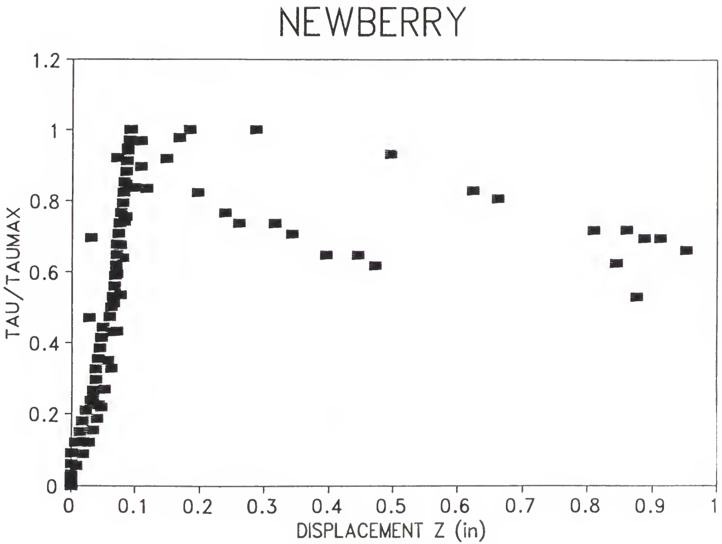


Figure 5.6. Semi-dimensionless T-Z curves, Newberry Site.

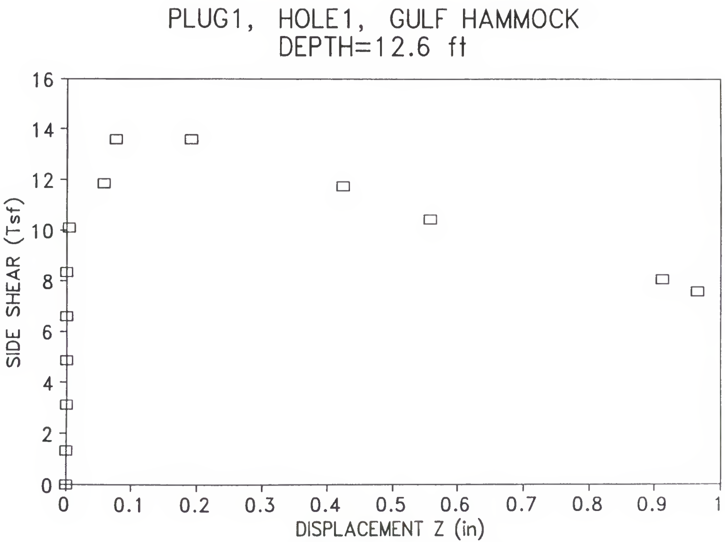


Figure 5.7. T-Z curve of plug #1, Gulf Hammock Site.

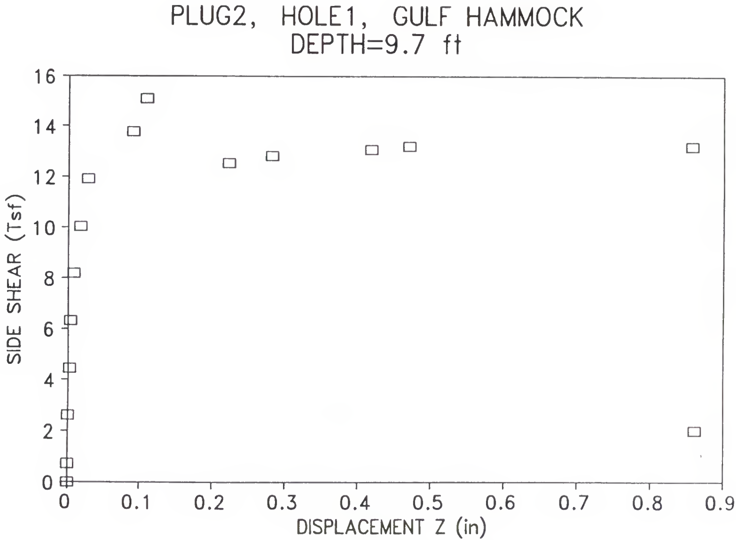


Figure 5.8. T-Z curve of plug #2, Gulf Hammock Site.

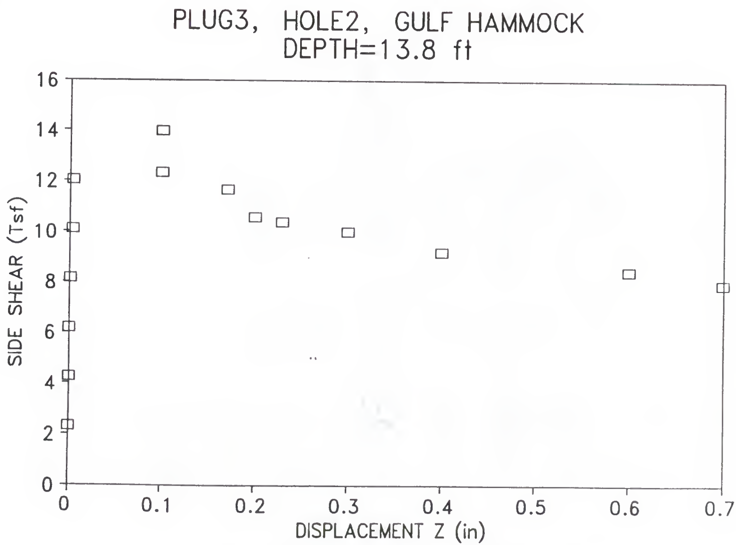


Figure 5.9. T-Z curve of plug #3, Gulf Hammock Site.

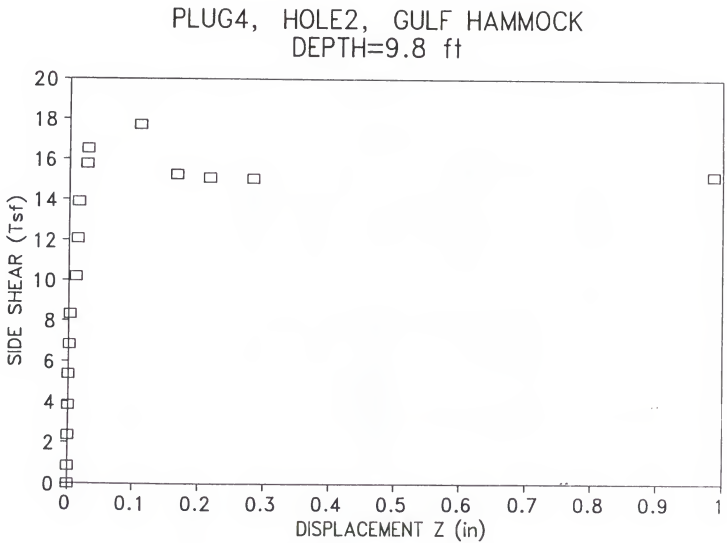


Figure 5.10. T-Z curve of plug #4, Gulf Hammock Site.

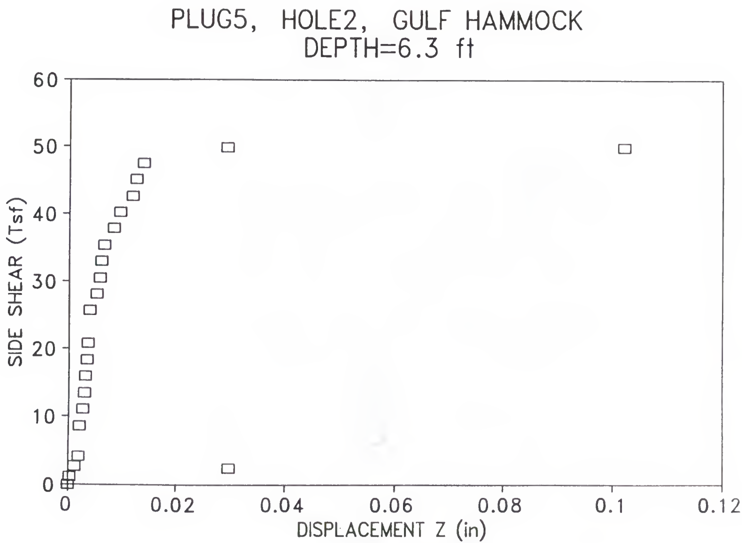


Figure 5.11. T-Z curve of plug #5, Gulf Hammock Site.

and the possible influence of the geostatic stresses presented before, are equally valid for this site.

E) Values of the secant slope corresponding to the maximum shear ranged from 140 to 178 tsf/inch for the more porous dolomite. A value of 499 tsf/in was found for the stronger one.

Figure 5.12 presents the obtained semi-dimensionless T-Z curves.

Miami Site

The results of the anchor tests (Figures 5.13 through 5.15) can be abridged as follows:

A) Maximum side shear stresses of 9.1, 12.3 and 24 tsf were computed for the three rocks that were tested. It must be pointed out that failure occurred through the grout in PT1; the 9.1 tsf value was based on the assumption that the anchor had a nominal diameter of 6 inches, and that the rock was able to sustain that shear stress, at least.

B) The displacement required to mobilize the maximum side shear varied from about 0.1 inches (inferred from the UF measuring system, plug PT2) to 0.56 inches. This last value is anomalous but, as explained before, the failure involved a weak grout (pullout PT1).

C) Values from 0.8 to 1 were found for the degree of softening. This is in accordance with the degree of waviness displayed by the (recovered) plugs. On the other

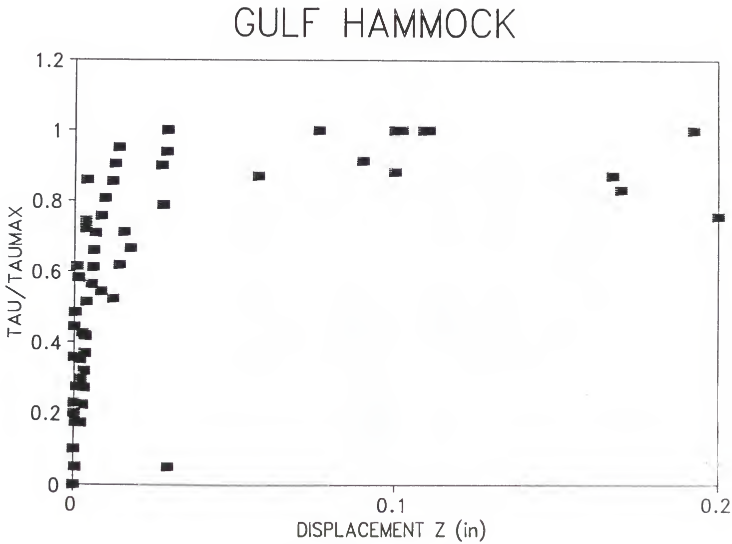


Figure 5.12. Semi-dimensionless T-Z curves, Gulf Hammock Site.

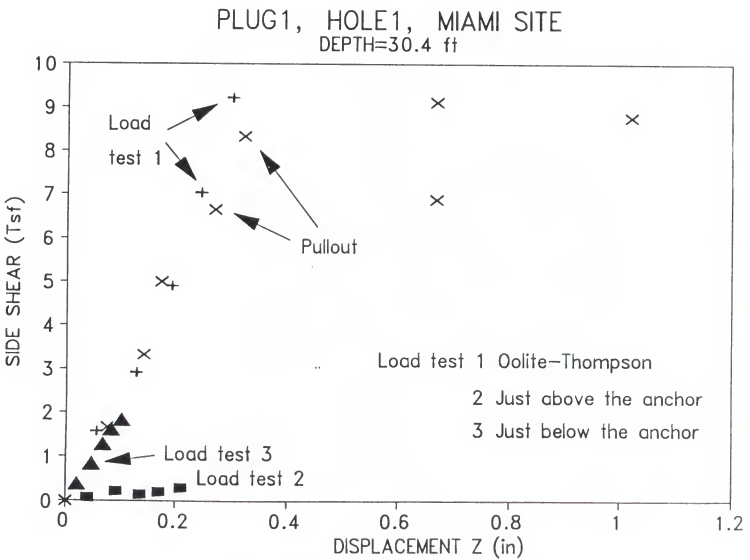


Figure 5.13. T-Z curve of plug #1, Miami Site.

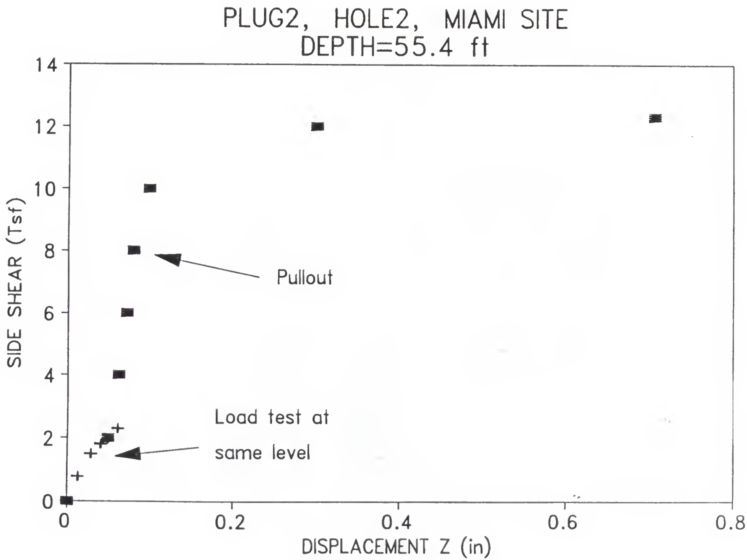


Figure 5.14. T-Z curve of plug #2, Miami Site.

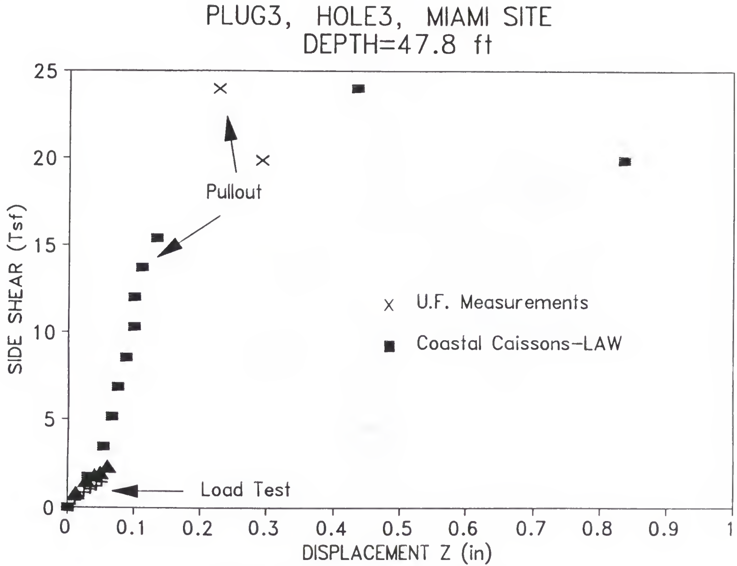


Figure 5.15. T-Z curve of plug #3, Miami Site.

hand, the plugs were installed at greater depths (larger than 30 ft), so any tendency to brittleness may have been counteracted by the geostatic stresses.

D) The secant slope corresponding to the maximum shear lies between 16 tsf/inch (abnormally low, perhaps due to failure of the anchor itself) and 120 tsf/inch. This last value is lower than those found at Newberry and Gulf Hammock, although the "intact" rock at the Miami Site (Fort Thompson) does not seem to be of lower quality. A hypothesis could be advanced to explain the situation, namely the possible effect of the mud. Unfortunately, no tests conducted in dry or drilled-with-water only holes are available to narrow the possibilities.

As stated previously, this series of tests had a distinct advantage, namely that T-Z curves found via small and large scale tests were available. Admittedly, each test was run on a different spot, thus bringing the spatial variability problem into the picture, and anchor PT1 failed through the grout, making the assessment of side friction somewhat uncertain; nevertheless, some comments may still be useful.

Figure 5.13 includes test shaft T-Z curves at levels located above and below PT1 anchor level. A remarkable similitude is apparent in one of the cases though the shaft T-Z curve corresponds to a different rock (the transition from the Miami Oolite to the Fort Thompson Formations). The curve corresponding to the stretch

located just below the anchor seems to match the initial part of the pullout T-Z curve. The other curve (stretch just above the anchor) looks somewhat anomalous.

Figures 5.14 and 5.15 show that anchor and load test T-Z curves are similar at low displacements. No comparison can be made beyond that point because little load shedding took part on the drilled shaft at those depths.

The semi-dimensionless T-Z curves appear in Figure 5.16.

<u>Miami Oolite Tests</u>

The tests collected on the Miami Oolite (Figures 5.17 through 5.22) yielded these results:

A) Maximum side shear stresses that ranged from 4.6 tsf to 7.2 tsf. The lengths of the anchors were similar in all tests (4.8 ft to 6.3 ft) so the possible effect of the L/D ratio could not be assessed. It must be pointed out that the rock shows a well developed network of large pores that could have been filled with the grout. Therefore, it seems feasible that the "true" anchor diameter could have been larger than the "nominal" one. The reported shear values may be considered to be "somewhat" high.

B) Displacements of 0.17 to 0.24 inches were needed to reach the maximum side shear. An apparently wrong value (0.38 inches) was recorded in one of the tests.

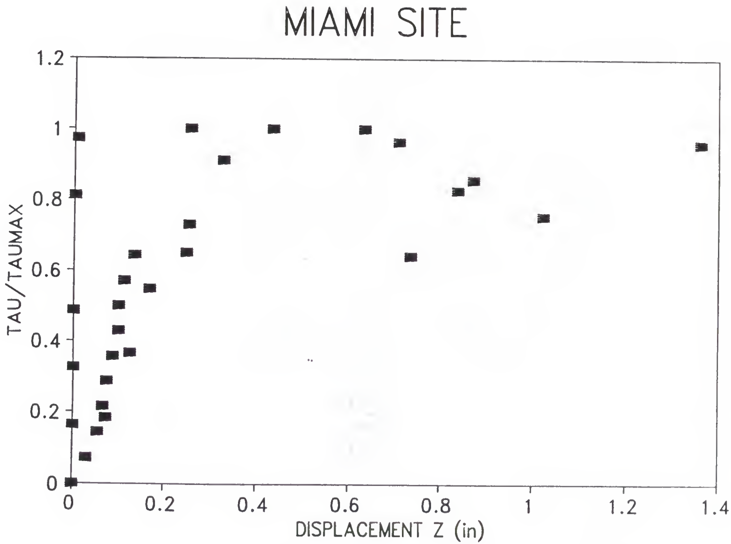


Figure 5.16. Semi-dimensionless T-Z curves, Miami Site.

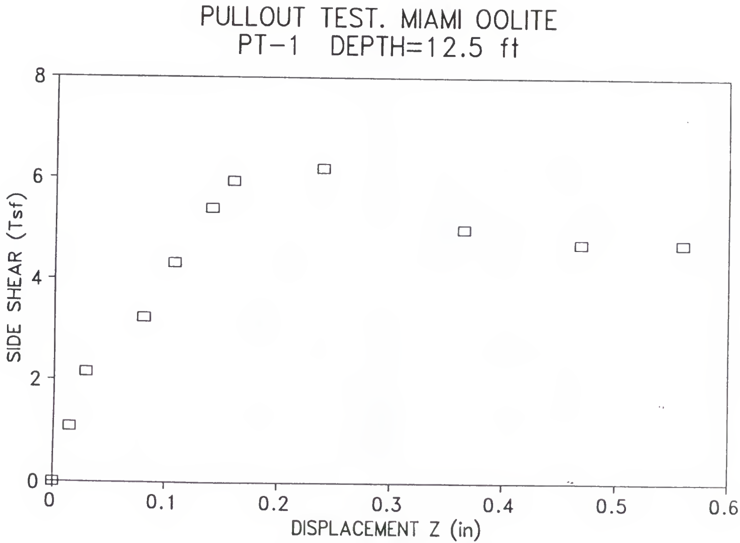


Figure 5.17. T-Z curve of plug #1, Miami Oolite.

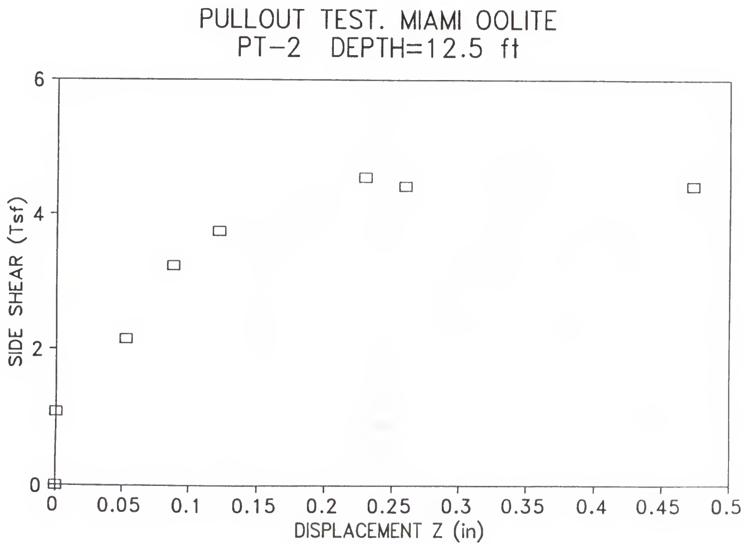


Figure 5.18. T-Z curve of plug #2, Miami Oolite.

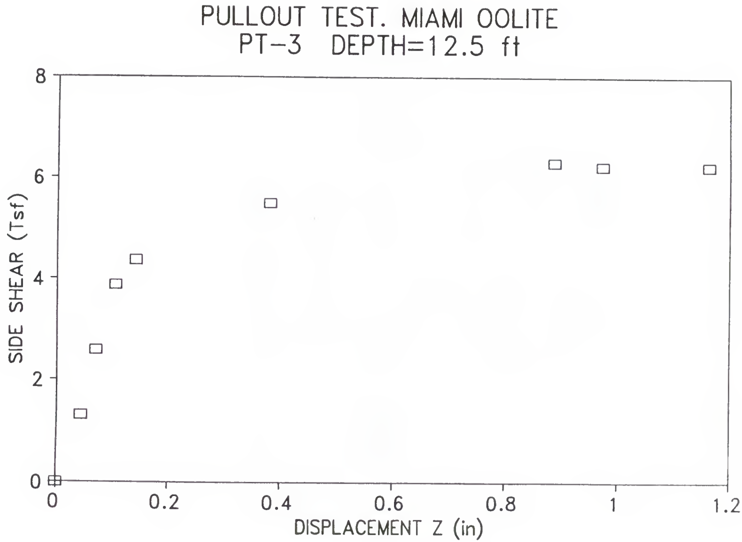


Figure 5.19. T-Z curve of plug #3, Miami Oolite.

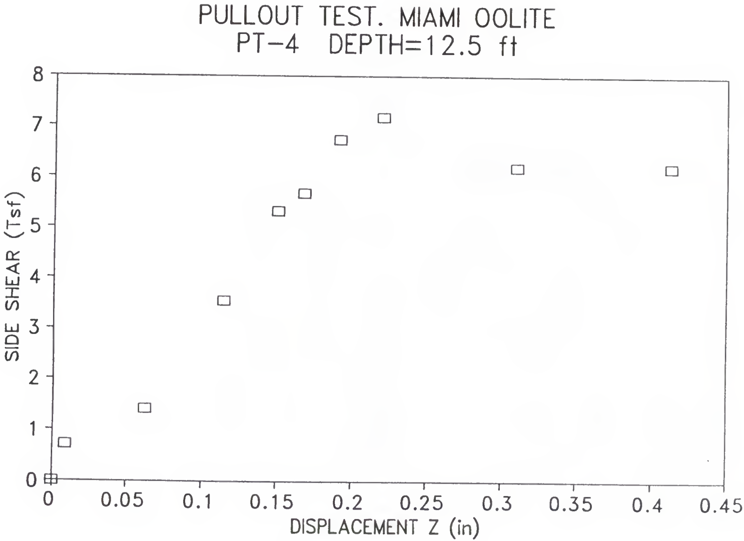


Figure 5.20. T-Z curve of plug #4, Miami Oolite.

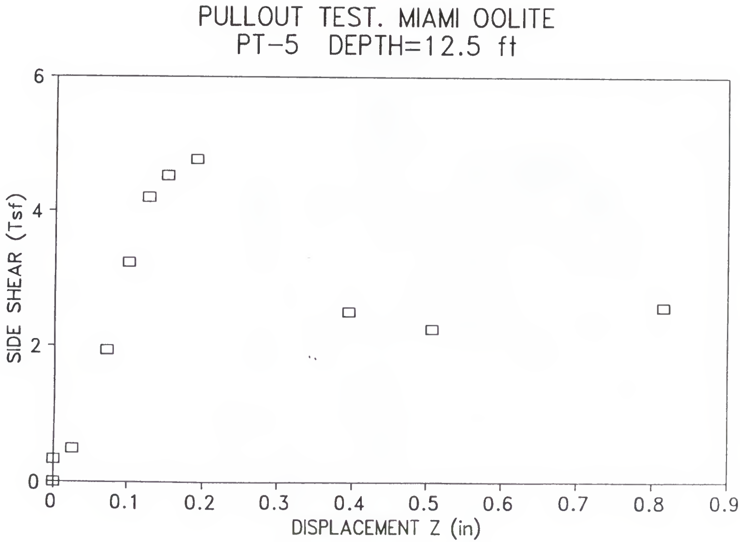


Figure 5.21. T-Z curve of plug #5, Miami Oolite.

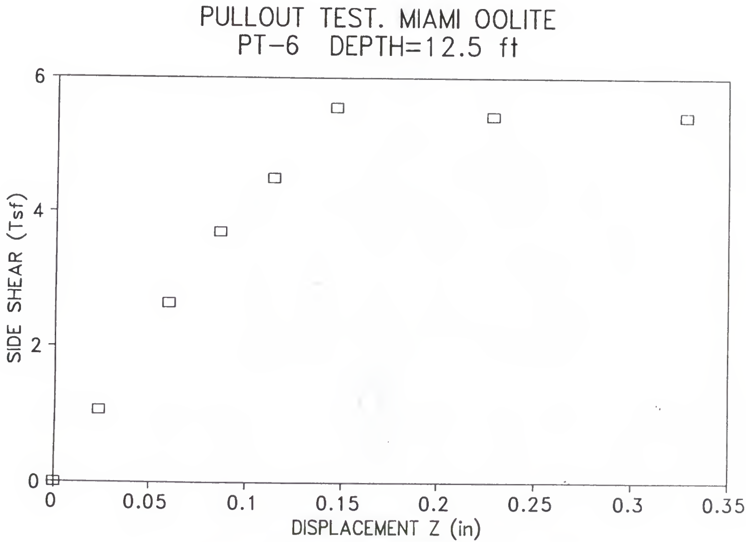


Figure 5.22. T-Z curve of plug #6, Miami Oolite.

C) Softening ratios ranging from 0.5 (somewhat low) to 1.0, with most of the values above 0.8. The visual inspection of the cores (with well developed dissolution features) gives the appearance that the anchor-rock interface must have been rough.

D) The secant slope values (at maximum side shear) ranged from 20 to 36 tsf/inch.

The semi-dimensionless T-Z curves can be seen in Figure 5.23.

Summary of Pullout Results

Table 5.1 summarizes the pullout tests results.

Table 5.1 Summary of Pullout Test Results

Anchor	length (ft)	Taumax. (tsf)	T-Z slope (tsf/in)	Displ. at Taumax (in)	Softening
Newberry					
2	0.6	26.9	220	0.15	0.5
3	2.8	21.2	75	0.30	0.7
4	1.2	30.1	167	0.20	0.6
5	2.3	25.3	275	0.10	0.8
Gulf Hammock					
1	1.2	13.6	178	0.08	0.6
2	2.0	15.1	145	0.11	0.8
3	0.8	14.0	140	0.10	0.6
4	2.0	19.5	176	0.11	0.8
5	2.0	50.0	499	0.11	--
Miami (Dade St)					
1	1.9	9.1	16	0.56	1.0
2	2.5	12.3	120	0.10[1]	1.0
3	1.6	24.0	109	0.22	0.8
Oolite					
1	6.3	6.2	26	0.24	0.8
2	6.3	4.6	20	0.22	1.0
3	5.2	6.3	7	0.38	1.0
4	4.8	7.2	33	0.22	0.9
5	5.2	4.8	25	0.19	0.5
6	6.4	6.1	36	0.17	0.9

[1] Inferred from measurements done via the UF technique

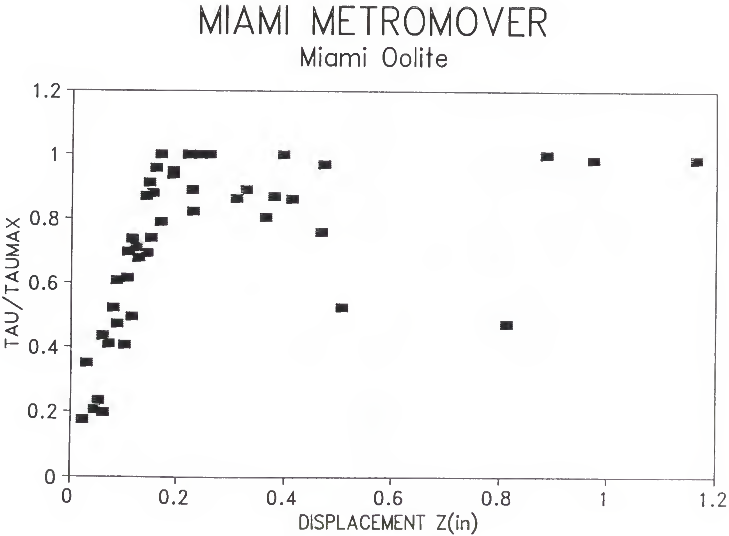


Figure 5.23. Semi-dimensionless T-Z curves, Miami Oolite.

The table shows maximum side shear values of about 5 tsf for the Miami Oolite, 25 tsf for the Ocala limestone formation, 10 to 24 tsf for the Fort Thompson and about 15 tsf for the porous dolomite at Gulf Hammock. An extreme value of 50 tsf was recorded for a finely crystallized dolomite at Gulf Hammock.

The displacements required to fully mobilize the maximum side shear ranged from about 0.1 to 0.5 inches. Sixteen of the eighteen cases had values between 0.1 and 0.3 inches. This seems to agree with previous findings from tests on full size sockets (Parra et al., 1990).

Figure 5.24 presents a summary of semi-dimensionless T-Z curves found from pullout an full size load tests (U.F. database). It appears that maximum lateral friction is mobilized at a movement of about 0.1 to 0.2 inches, independently of the size of the socket.

The degree of softening ranged from 0.5 to 1; 4 out of the 18 cases fell in the 0.6 to 0.8 range and 11 cases showed values larger than 0.8. According to the results obtained at Newberry and Gulf Hammock, the softening tends to decrease, at a modest rate, as the length of the plug increases. It may be worth pointing out that a prevailing trend towards a "ductile" type of behavior was apparent in T-Z curves computed from full size load tests.

The results found at Newberry and Gulf H. tend to indicate that the plug aspect ratio (length/diameter) may

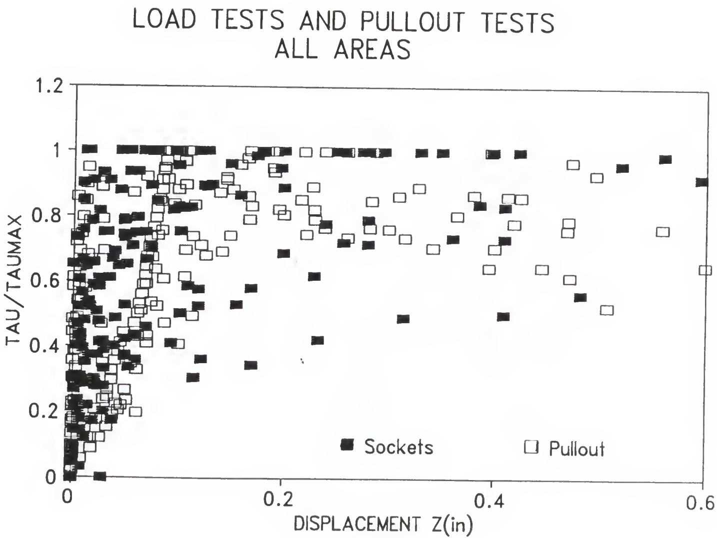


Figure 5.24. Summary of semi-dimensionless T-Z curves

affect the computed side resistance to some extent. The difference in maximum side friction computed from short anchors (L/D values from 1.5 to 2) and longer ones (L/D values close to 6) was of the order of 20% to 26%. Based on the results obtained at Gulf Hammock, it appears that longer plugs lead to larger side friction values; this is in accordance with trends reported by Rowe and Pells (1980). Smaller side shear values are to be expected if the L/D ratio is increased from about 4 to 10, according to the above mentioned investigators.

Lab Seismic Test Results

A total of eighty two pieces of rock were tested for both P wave and S wave speeds, using impact as well as high frequency excitation on most of the cores. Each test was repeated at least three times. Practically all the cores recovered at Newberry (total of 41) were examined. Only those recovered at the pullout depth intervals were tested for the other sites (24 at Gulf Hammock, 11 at the Miami site, and 6 still available from the Miami Oolite). A summary of the results is shown in Appendix D.

The experience collected throughout the testing program indicated the following:

A) The determination of body (P) waves was simpler, as expected. The results were very reproducible, the arrival times being practically independent of the type of excitation (high frequency or impact). A concrete tester type of device seemed to be the easiest one to use.

B) The determination of arrival times for shear (S) waves was indeed more difficult and in many cases uncertain, even if one knew the P wave arrival time in advance, by performing this test first. Pairs of tests with leading waves of different sign were performed in an attempt to get rid of the P wave. The results were ambiguous in most of cases.

It should be pointed out that the so-called cross-over technique, as suggested by Robertson et al. (1986), could not be used in the present case, because the accelerometers that were utilized did not have the same frequency response. A pair of very sensitive accelerometers, recently acquired by the UF Geotech Department, could be used for future research.

C) The technique seemed to serve a useful purpose: it may allow one to ascertain the degree of homogeneity of the cores. This may help in the process of deciding which samples can be gathered for compression and tension tests, for example.

A summary of results for each of the sites will be presented in the following paragraphs.

Newberry Site

Table 5.2 displays the P wave speeds for several depth intervals.

TABLE 5.2 Newberry Site.
Summary of Lab P wave Speeds in ft/sec

0 to 5 ft (HOLES 1, 6 7 AND 8)

#1	#6	#7	#8	
6674	8325	4679	10598	AVE= 7700
6976	8797	6320	5036	STD= 1917
11107	6918	5974		
	8445	6548		

TABLE 5.2 Continued.

0 to 5 ft (HOLES 1, 6 7 AND 8)

#6	#7
5505	9449
9374	
9946	

5 TO 9 ft (HOLES 4, 6, 8 AND 7)

#4	#6	#8	#7	
9431	5128	6280	8756	AVE= 7374
7246	7476	7018	8645	STD= 1210
6865	6892			

---

9 TO 13 ft (HOLE 7)

#7
8672

---

13 TO 18 ft (HOLES 1, 2 AND 7)

#1	#2	#7	
7389	6971	8970	AVE= 7233
9505	6729		STD= 1440
	5880		
	5189		

AVE= Average; STD= Standard Deviation

According to the table, 7400 ft/sec could be considered a rough average for the entire profile. The occurrence of large variations from site to site or in a given hole, is noticeable. It must be pointed out that the anchors were set in the 0 to 9 ft and 13 to 18 ft intervals, where the amount of information is larger. The test results were indeed useful as a guide to ensure that similar cores were used for compression and tension tests.

Figure 5.25 shows field and lab seismic results. Although the scatter is large, it appears that there is not a marked difference between them. This suggests that the properties of the rock mass and the cores may not be very different from each other.

Gulf Hammock Site

Body P wave speeds corresponding to the pullout stretches are shown in Table 5.3.

Table 5.3 Gulf Hammock.
Summary of Lab P wave Speeds in ft/sec

5 to 9 ft (HOLE 2)

#2	
13945	AVE=11550
11576	
9133	

9 to 15 (HOLES 1 AND 2)

#1	#2	
5513	5412	AVE=6100
6084	5283	STD= 400
5741	6496	
5971	6366	
6058	6487	
6614	5942	
	6471	
	5847	
	6616	
	6412	
	6448	

The highest values (4 to 9 ft stretch) correspond to a much stronger dolomite stratum; the same type of rock was found in borehole #1. The second group (9 to 15 ft) corresponds to the more porous rock. It appears that this site is somewhat more uniform than Newberry, as judged from the standard deviation.

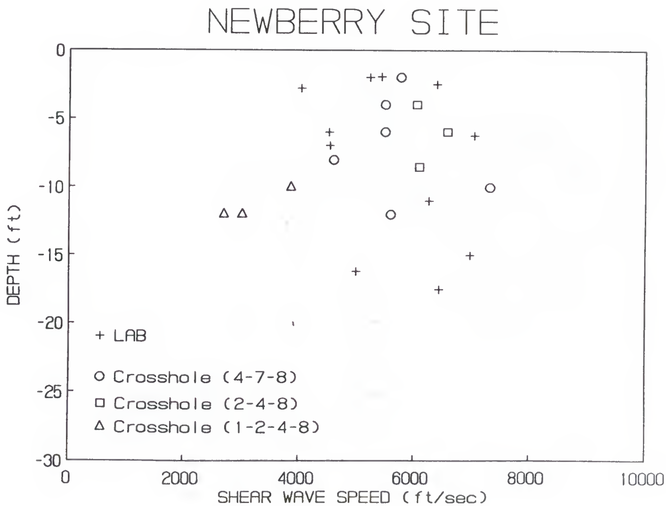


Figure 5.25. Summary of lab and field seismic tests, Newberry Site.

It should be pointed out that the lab seismic tests were used as a guide to ensure that similar cores were used for compression and tension tests.

Miami Site

The tests on the cores were run in two stages (Table 5.4):

A) One series was performed using the torsional (impact) scheme only. All cores were inspected, excluding sample 2-3, because it had not been trimmed.

B) A second run was done, using the high frequency tester. Unfortunately, most of the cores had already been tested for strength (unconfined compression and split tensile tests), so they were not amenable for seismic tests.

Although the information is scarce, it appears that:(a) the core used for the split tensile in the first group (pullout PT1) could have been stronger than the others, and (b) "similar" specimens were used in PT3. Not all the cores were tested for the remaining case; therefore, no comments can be made.

Table 5.4 Miami Site. Lab Seismic Tests

SAMPLE #	DEPTH (ft)	IMPACT Vp (ft/sec)	IMPACT Vs (ft/sec)	HIGH FREQUENCY Vp (ft/sec)	HIGH FREQUENCY Vs (ft/sec)	TEST
1-1	27.5		7939			C
1-2	|			9860	7386	
1-3	32.5	>12000				T
2-1	53.5	11609		11240	7436	

Table 5.4 Continued.

SAMPLE #	DEPTH (ft)	IMPACT Vp (ft/sec)	IMPACT Vs (ft/sec)	HIGH FREQUENCY Vp (ft/sec)	HIGH FREQUENCY Vs (ft/sec)	TEST
2-2	|	13055				T
2-3	58.0					C
3-1	46.5		7272			T
3-2	|		6519			C
3-3	49.5		5637	8515	6199	

C=Unconfined compression; T=Split tensile

The wave velocities (P wave speed of the order of 10000 ft/sec or larger) are somewhat larger than those found for the other areas, excluding the strong dolomite stratum at Gulf Hammock.

Miami Oolite Tests

As explained before, the seismic tests were performed on samples different from the ones used for compression and tension tests. Therefore, it is not possible to ascertain whether "similar" cores were used for each test. The tested cores indicated P wave speeds from 3000ft/sec to about 4500 ft/sec, thus showing that this was perhaps the weakest of the investigated rocks.

Destructive Tests. Correlations with Lab Seismic Results UF Research Sites

Unconfined compression and split tensile tests were performed on selected cores recovered at Newberry and Gulf Hammock.

A correction proposed by ASTM was applied to the compressive strength, to take the length/diameter ratio of the samples into account. The obtained results have been summarized in Appendix D.

Stress-strain curves were found for some of the cores (the longest ones). The tests showed the following:

A) Initial tangent elastic moduli of about 250000 tsf (Newberry) and 30000 to 200000 tsf (Gulf Hammock., porous and dense rocks, respectively). These values are about 0.1 to 0.7 times the elastic modulus of the 928 grout.

B) Modular ratios, i.e. ratio of Young modulus to uniaxial compressive strength, of the order of 300 to 800 were computed for the Gulf Hammock site. Rather large values (1780 and 2140) were computed for two Newberry samples. Figure 5.26 contains strength and moduli results corresponding to the present research and also to areas where research on rock sockets has been conducted. It seems that the analyzed cases fall near the boundaries shown in the Figure.

Table 5.5 shows the elastic moduli values found from compression tests.

Table 5.5 Elastic Modulus. Cores Tested in Compression

Sample	Elastic Modulus E (tsf)	P wave Vp(ft/sec)	Uniaxial Strength (tsf)
Newberry 1-5	281800	11107	156
" " 6-7	233400	9374	106

Table 5.5 Continued

Sample		Elastic Modulus E (tsf)	P wave Vp(ft/sec)	Uniaxial Strength (tsf)
Gulf H.	2-12	45000	6616	53
"	2-22	29630	5412	46
"	2-7b	162361	11576	512
"	2-7c	225746	13945	677

A search for possible correlations between rock compressive strength and the P wave speed was performed. Figures 5.27 and 5.28 indicate that the lab seismic tests may yield information on the unconfined compressive strength of the rock. The plots seem to indicate that an exponential relationship between the elastic modulus (the P wave speed is proportional to the square root of the former) and the strength may exist. The correlations are local, of course, but could be validated for other types of geological formations.

A simple parametric exercise was performed to compare elastic moduli at small strains (seismic) to those found by conventional destructive tests. The material was assumed to be isotropic, with Poisson ratio values from 0.15 to 0.3. The following expression was utilized to compute the elastic modulus (E):

Vp^2 = (L+2G)/RO, where

L = nu*E/((1+nu)(1-2*nu)); 2G = E/(1+nu)

nu = Poisson's ratio; E = Elastic modulus

Vp = P wave speed.

Values of Vp corresponding to the samples on which the compression tests had been done, were used as input; Table 5.6 shows the results.

Table 5.6 Elastic moduli (tsf) for
Selected P wave Speeds (ft/sec)

nu	Vp=11107	9374	6616	5412	11576	13945
0.15	241000	171600	85500	57200	262000	380000
0.25	212000	151000	75200	50300	230000	335000
0.30	189000	135000	67000	45000	205500	298200
Measur-ed	281800	233400	45000	29630	162361	225746

As expected, the moduli computed from P wave data are larger than the values found from the compression tests. The Newberry cases are an exception, thus indicating that either there are errors in the measurements or that the rock may not be isotropic.

Correlations between Pullout and Lab Test Results

As stated in Chapter 3, one of the primary aims of the research was to complement and validate current correlations between the maximum side shear, found from pullout or load tests, and the strength of the rock, obtained from unconfined compression and/or split tensile tests.

Maximum Side Shear-Unconfined Compression correlation

The information gathered throughout the research is shown in Figure 5.29. The graph contains results found from pullout and load tests, and covers various Florida calcareous formations. It must be pointed out that tests with only unconfined compression data are far more common than those

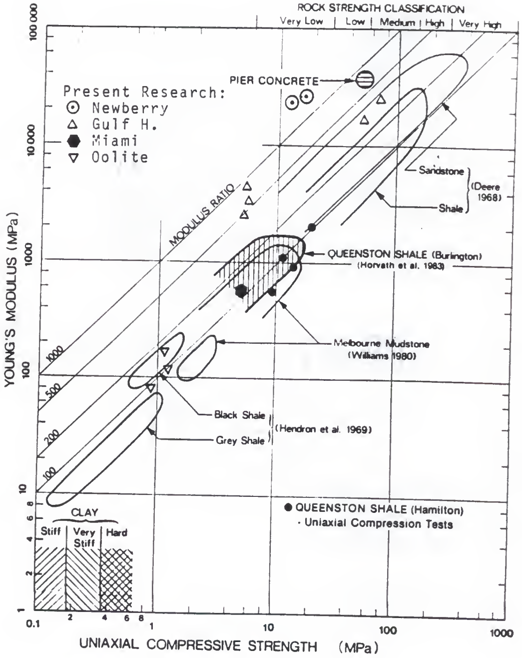


Figure 5.26. Rock unconfined compressive strength-elastic modulus. Includes data from present research and from other sites where rock sockets have been studied by other researchers. Adapted from Horvath and Chae, 1986.

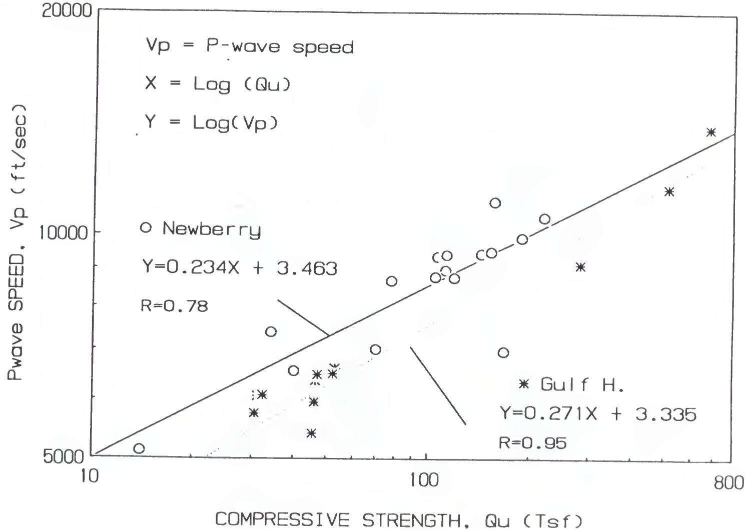


Figure 5.27. Correlation between rock unconfined compressive strength and P-wave speed, Newberry and Gulf H.

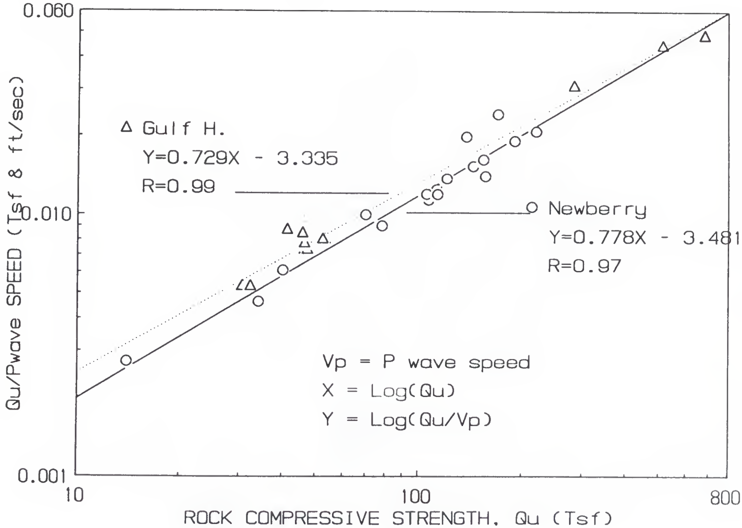


Figure 5.28. Correlation between rock compressive strength and the strength/P-wave speed quotient, Newberry and Gulf H.

with lab compression and tension tests. The results indicate that:

A) The available guides, which by the way represent a variety of rocks, are not substantially different. It should be pointed out that the one adopted by Gupton and Logan (originally proposed by Pells) is based on load tests performed on East Florida formations (Anastasia, Fort Thompson, Miami and Key Largo).

B) Some of the points lie below the lowest (most conservative) of the existing curves. The corresponding cases involve shafts drilled with mud (Jax, Tampa hospital) and without mud (Tampa airport). On the other hand, several several shafts cast under mud gave results that lie above the current criteria. Therefore, any conclusions on the effect of this variable could be premature.

Most important, it must be pointed out that the strength values used to find the Alpha coefficient came from tests on cores recovered in holes different from the test hole. Typically, results from the closest and most representative holes, as judged by the company that performed or reported the test, were used. The spatial variability of the rocks makes it difficult to guarantee the extrapolation of values from one location to another, though. The pullout tests performed at Newberry, Gulf Hammock and Miami sites are an exception. Cores recovered at the anchor level were used for the tests.

C) All data found from pullout tests are located close to or above the lowest (Williams et al., 1980) curve.

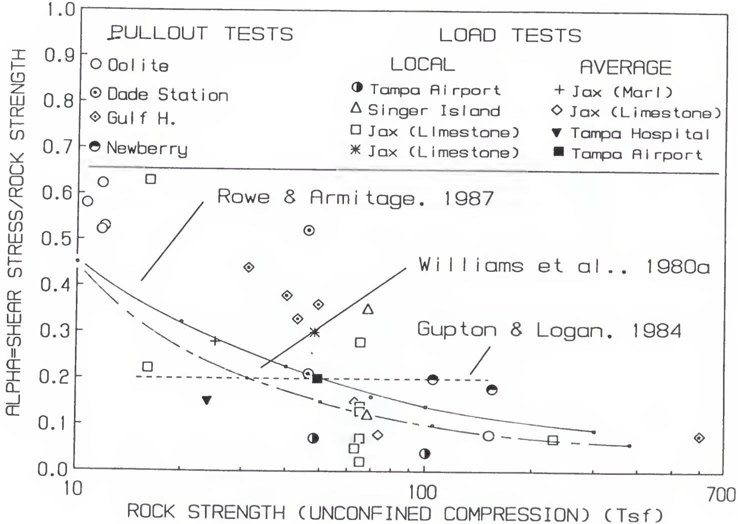


Figure 5.29. Maximum socket side friction as a function of the rock unconfined compressive strength. Existing guidelines and new data.

Maximum Side Friction and the Rock Strength Envelope

Recently, McVay (1991) proposed a new approach that considers the failure envelope of the rock, defined by the friction angle and the cohesion. The predictive method is also based on the premise that low radial stresses (assessed via FEM modeling by the mentioned researcher) occur along the socket when the friction is mobilized.

At least two tests, performed on similar rock samples are required to define the failure envelope. A simple combination, namely a split tensile and an unconfined compression test, seems to be appropriate for two reasons: (a) it covers the expected (low) normal stress range and (b) circumvents the need for specialized rock triaxial equipment. One special assumption has been made at the onset, i.e., that the possible effect of the pore water, if present, is considered to be minimal.

Three current approaches can be invoked to define the envelope:

A) Consider that the state of stresses corresponding to the tensile test is given by: minimum principal stress=computed tensile stress, and maximum principal stress=0. The envelope can be found as the line tangent to the corresponding Mohr circle and the circle obtained from the compression test.

B) According to elastic theory, the maximum principal stress in the split tensile test is compressive and equal to three times the computed tensile stress. The new

Mohr circle and the one corresponding to the unconfined compression test can be used to trace the envelope.

C) Adopt the criteria proposed by Hoek (1983), which uses the compressive and the tensile strengths to fit a curved envelope.

A way was devised to combine the value of shear strength at null normal stress, the recorded maximum side shear stress, the compressive and the tensile strength in a dimensionless way; this allowed him to put the three approaches, as well as the results from pullout tests, together. The obtained values, including those reported by McVay (1991), have been summarized in Figure 5.30. Some points deserve attention, as follows:

A) The three approaches to fit the envelope tend to coincide for values of the compressive strength/tensile strength ratio larger than about 10, as one would expect. McVay's and Hoek's envelope fitting techniques give similar results in the stress range of interest (low normal stresses) for a wide range of the ratio above mentioned. The other method yields larger shear strength values for compressive/tensile strength values lower than about 8.

B) The vast majority of the pullout test results lie above or close to the presented criteria. This means that the normal (radial) stress on she socket-rock interface is almost null (as predicted by McVay, 1991) or somewhat larger than zero, thus rendering the predictive method conservative. The disagreement seems to be specially

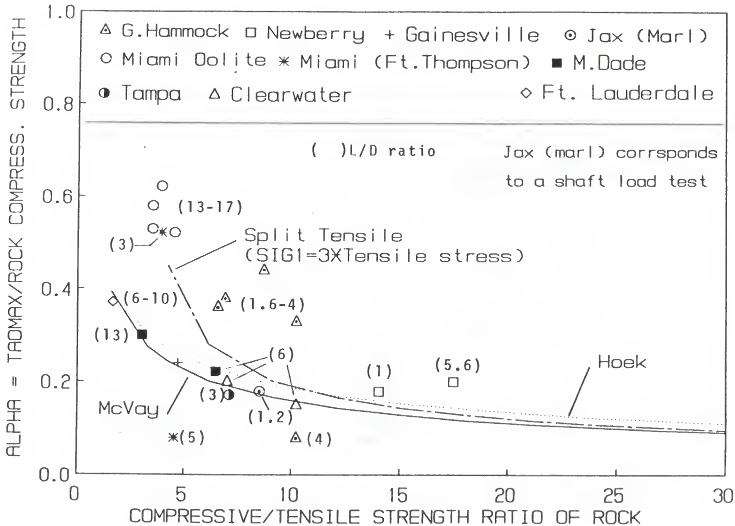


Figure 5.30. Maximum socket side friction as a function of the rock's shear strength at null normal stress. McVay's guideline and new data.

serious if the Miami Oolite results are considered; nevertheless, there are reasons to believe that the computed Alpha value could be smaller (see T-Z behavior in this Chapter). As stated previously in this Chapter, there are reasons (Rowe and Pells, 1980) to infer that smaller side shear values (and Alpha values) would have been found in Gulf Hammock, should longer anchors had been tested. As depicted in Figure 5.31, long anchors (L/D of the order of 10 or more) as well as sorter ones (L/D of the order of 5) yielded results that tend to fit McVay's guideline; it appears that more research is needed on this topic.

The above presented findings, namely the condition of "almost null radial stress" seems to disagree with results reported by other researchers (Donald et al, 1980; Williams et al., 1980b).

C) Only two points were located under the prediction lines. As the occurrence of an average normal (radial) tensile stress seems unreasonable, other explanations must be sought.

A first reason could be that different types of rocks were tested under compression and tension. This could have been the case of pullout PT2 (Miami site, see page 109 of this dissertation). The selection of cores was carefully done for the other case (Gulf Hammock) so this item is discarded as an explanation for the observed behavior. The effect of the length of the anchor can be invoked as well. According to findings presented by Rowe and Pells (1980),

longer anchors should yield a larger shear stress. It should be pointed out that the anchor and the rock exhibit similar elastic moduli for the case at issue. Another hypothesis could be put forward, namely that the failure occurred through the anchor before the rock failed. No conclusive comments can be stated because the anchor could not be recovered.

Correlations with Lab Seismic Results

The fact that the results of seismic tests (wave speeds) are essentially related to the deformability properties of the rock, may indicate that a correlation between the slope of the T-Z curves and the P wave speed could be searched for.

The P wave speed was selected because, as explained earlier, it was easier to find, and was considered to be more reliable. In any event, P and S wave speeds are related to each other. The secant slope corresponding to the maximum side shear was chosen as the other variable. Considering that the T-Z curves do not seem to display any trend to resemble a simple curve (say a hyperbola), it may be appropriate to aim at simpler fits. A piece-wise one is the easiest: a first stretch with the slope above defined, up to the maximum side shear level, followed by a sudden drop to a side shear equal to 0.7 to 0.8 times the maximum side shear, and a horizontal line from that point on.

Figure 5.31 shows the correlation. A trend seems to exist if the results from Newberry and Gulf Hammock are considered; rocks with larger P wave speeds lead to stiffer T-Z responses. The Miami Oolite and the Fort Thompson (Miami site) data fall in other regions of the plot, thus indicating that the relationship is site or formation dependent. The results from the Miami site show a larger scatter which may be due to the fact that the tests were done in different strata of the Fort Thompson formation; on the other hand, the smallest of the values (case of PT1) corresponds to a case of failure of the grout, and may be considered somewhat questionable.

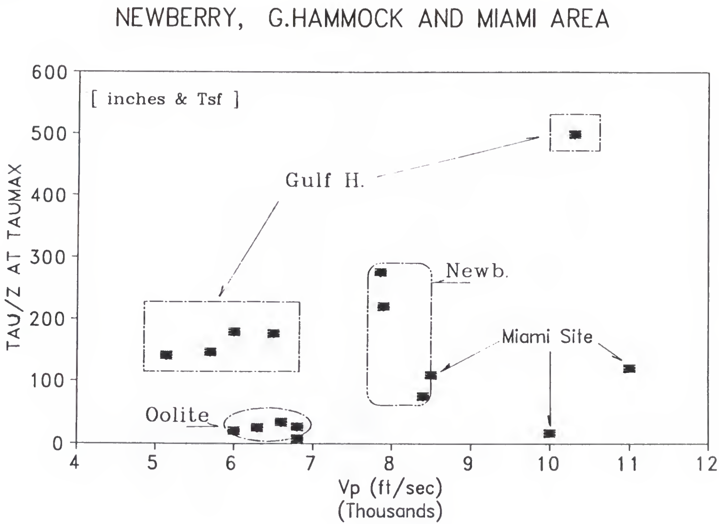


Figure 5.31. Correlation between the secant slope of the T-Z curve (at maximum side shear) and the P-wave speed found in the lab.

CHAPTER 6
CONCLUSIONS AND RECOMMENDATIONS

Research has been performed to update and improve the rock anchor pullout technique, which is sometimes used to define the maximum side shear, required to design a shaft socketed in rock. The research also measured how side shear develops as the anchor moves (the T-Z curve).

There seems to be a lack of information about how the vertical stress changes in a rock mass due to the movement of a shaft (or an anchor). Some Finite Element runs were conducted to assess the applicability of the T-Z approach in rock sockets.

Previous research had shown a large scatter in correlations between the rock properties commonly found during the exploration stage (standard penetration blow count, recovery indexes) and the maximum side shear offered by the rock. On the other hand, the literature search indicated that the available drilled shaft design methods are to a large extent empirical and therefore site dependent. The validation of such methods had to be attempted.

Thirteen pullout tests were performed at three sites with different geological formations, using various

techniques to cast the plugs and measure their displacement under load. New data on tests performed in the Miami Oolite was also obtained. Experience was gained on the performance of the methods used to form the anchors and on the systems utilized to record displacements during the pullout.

An attempt was also made to use non destructive lab seismic techniques and to correlate these results with those found from lab destructive (strength) and pullout tests.

Some conclusions were arrived at, as follows:

A) Literature searches performed for this and for previous research showed that the available shaft design methods are essentially empirical as regards the determination of the maximum side shear resistance of the rock. Two of them (C.I.R.I.A. and McMahan) use the SPT blow count, while the others resort to the unconfined compressive strength of the rock. The settlement problem is handled by FEM elastic and sometimes more complex nonlinear solutions. Both require that the elastic moduli of the intact rock and of the rock mass be known. Little information is available on these two parameters, specially the second, for Florida rocks. The T-Z approach circumvents this problem, and therefore emerges as a convenient alternative for predicting the behavior of rock sockets under axial static loading.

B) The preliminary numerical modeling indicated that the change in vertical stress term may not be important for the case of rock sockets. Therefore, the assumption of negligible vertical stress gradient used for piles in soil seems to be valid, and the T-Z method approach may be applicable to the case of drilled shafts installed in rock.

C) Some commercial grouts were tested for shrinkage, fluidity and strength after 3 to 5 days of curing. The products displayed excellent behavior: no shrinkage was apparent; they flowed with no trouble though 1 and 1.5 inch pipes; they remained fluid for over 15 minutes after the mixing process was completed and reached strengths of 4000 psi and higher after the 3 to 5 day curing period.

D) Two techniques were used to cast the anchors, namely: (a) the conventional tremieing procedure and (b) a new one (developed for this research), in which the grout was held in a container and lowered to the required depth. Both schemes were satisfactory, provided some preliminary measures were taken, as indicated in the next two conclusions.

E) The hole should be back-filled with grout or some other material to the level of the bottom of the anchor, prior to casting the plug. This preliminary step would avoid possible leaks through the bottom seal (case of the UF container method) and possible contamination of the grout (tremieing method), as observed at the Miami

site. The volume of grout being poured (tremieing method) should be monitored.

F) A procedure in which the anchor reinforcing cage is set in the hole prior to conducting the tremieing,should be preferred. Some contamination may occur if the insertion is done afterwards; the cage may drag some dirt and mud as it goes down in the hole.

G) The caliper designed for this project proved to be simple to use and accurate. The information on the profile of the hole provided invaluable guidance in choosing the testing stretches and the amount of grout needed for a given anchor length (Gulf Hammock site). The match between expected and as-built plug lengths is encouraging. The records made at the Miami site compared quite well with the measurements made on one of the recovered plugs.

H) The new system used to monitor the movement of the plug proved to be simple and accurate. A comparison betwee this scheme and the one in which the movement of the Dywidag bar is monitored by dial gages was made at the Miami site. Both gave similar results. Nevertheless, the UF technique displays three distinct advantages: (a) it is not affected by temperature variations; (b) it is not affected by the elongation or slippage that may occur at the Dywidag couplings, and (c) quick changes in displacement (at the moment of failure) can be handled with no problem.

I) The pullout test appeared to be simple to perform and reproducible. Tests performed in similar rock strata (Gulf Hammock site) yielded similar side shear values.

J) It appears that the anchor aspect ratio (length/diameter) may affect the computed maximum side shear. Changing the aspect ratio from 1 to 6 led to 20%-26% variations in the computed maximum side shear, at least for the tested rocks.

K) The T-Z curves obtained from small scale (pullout) and full scale load tests were compared for the Miami site. The comparison showed that the initial parts of the full-size and the pullout T-Z curves seem to agree with each other. No comparisons could be made at larger displacements because little load shedding took place at the corresponding depths. These findings seem to support the basic assumption that the diameter of the hole does not affect the side shear behavior, at least for diameters of 6 inches or larger.

L) The displacement required to mobilize the maximum side shear fell between 0.1 and 0.2 inches for the majority of cases. This range is similar to that found for full size load tests in earlier research, indicating that the size effects, 6 inch pullout vs. 24 inch or larger rock socket, may not be significant.

The degree of softening (side shear at larger displacements/maximum side shear) was in most cases larger

than 0.8. This figure agrees with values found via full size load test, thus indicating that the scale effect may not be especially notorious.

M) The obtained Alpha values (pullout maximum side shear divided by the intact rock unconfined compressive strength) were compared to the existing guidelines (Williams et al, Rowe and Armitage, Gupton and Logan). The new data fell either close to or above the lowest of the curves (Williams et al) thus indicating that this guideline could be appropriate as a <u>starting point</u> for a predesign.

N) The evidence seems to support the new predictive method proposed by McVay. Only two of the eighteen tests yielded maximum side shear values that fell under the predicting line. The spatial variability of the rocks in one of the cases and the possibility of experimental errors in the other could be invoked to explain the disagreement. Nevertheless, more research is warranted to ascertain the effect of the anchor Length/Diameter ratio on the computed side shear.

It must be pointed out that the basic idea behind McVay's method is sounder because the failure envelope of the rock is taken into account.

O) The information provided by the non-destructive lab seismic tests may be very useful. Every single core could be tested thus providing guides on how to set "similar specimens" aside for different tests. The

determination of the P-wave appears to be simpler and more reliable than that for the S-wave. Concrete testers, available in the market, can be used for the purpose.

P) The results indicate that there is a relationship exists between the P-wave speed and the unconfined compressive strength of the rock cores. The correlation could be used to predict the strength of rock pieces that may undergo other types of tests, say split tensile.

Q) The P-wave speed appears to be related to the secant slope of the field T-Z curves. The relationship was proven to be site dependent.

The recommendations can be stated as follows:

A) The numerical modeling of a broader set of cases must be undertaken to verify the applicability of the Load Transfer Approach to the analysis of rock sockets. Various combinations of parameters such as shaft length/shaft diameter, ratio of elastic moduli (Erock/Eshaft) as well as the use of approaches that consider failure, like the one applied by McVay, should be considered.

B) The pullout test technique, specially the installation of the anchor, needs to be standardized. Some items that deserve attention are:

* Some type of device should be used to get an idea about the geometric profile of the hole. The caliper developed for this research may serve as a starting point; the scheme should be improved so that the

orientation of the device can be controlled as it travels the hole.

* The hole, excluding the testing stretch, should be cased. The occurrence of cave ins tend to interfere with any preliminary operations, say the insertion of a caliper, the setting of the tremie pipe, or the insertion of the anchor itself. The experience gathered at the Miami site clearly showed that the casing and the borehole should be advanced almost <u>simultaneously</u>. This type of procedure may circumvent the need for using drilling mud; the presence of mud, coupled to other factors, as explained earlier in this document, may lead to <u>the contamination of the grout</u>.

* The characteristics of the drilling mud must be monitored, should the casing method be impracticable. The monitoring should be performed just before and after the flushing process. The anchor must be installed with no delay to prevent the formation of a cake. This cake may not be removed by the conventional flushing.

* The need for filling the hole underneath the testing level should be stressed. This step may avoid unnecessary delays and uncertainties during the grout pouring process.

* Some attempts should be made to verify the top level of the anchor, as soon as the grout has set. Supplemental measures can be taken should one find that

nominal and as-built lengths differ from each other.

* The technique that was developed for measuring the displacement of the anchor is simple, inexpensive and less prone to errors. It is recommended that the scheme be utilized for future tests. A thicker Kevlar rope should be used.

C) It is recommended that some tests be performed to verify the possible effect of the drilling method on the T-Z behavior. Holes originally bored with a 4 inch core barrel could be enlarged with a reaming tool, like the one used at the Miami Site. The conditions found at Gulf Hammock (rock strata with rather uniform stretches and no marked tendency to develop karstic features, at least to the explored depth of 16 ft) make this an ideal site for this type of research. The possible effect of the drilling mud could be studied as well. Some more tests could be conducted to complement the information about the effect of the length of the anchor on the side shear resistance.

Any consistent and well documented progress in this aspect would mean a big leap towards the solution of a crucial point: can the pullout test eventually replace the full size load test?

D) It is suggested that more pullout tests be conducted to complement the database collected so far. Some regions, and corresponding geological formations, need better coverage: Jacksonville, Naples, and Tallahassee,

for example. Attempts should be made to perform pullout tests at the same site (ideally the same spot) where full size load tests are to be conducted.

E) It is recommended that a thicker Kevlar rope be used in the system that measures the displacement of the anchor, because smaller sizes tend to fray.

F) It is recommended that lab seismic testing be continued on the remaining Gulf Hammock cores. The new set of high sensitivity accelerometers could be used to assess the applicability of Campanella's cross-over technique for S-wave determinations. The results could be compared to the values found using torsional impact.

G) It may be time to begin some research on the tip (end bearing) component. A simple setup (the sketch is shown in Appendix A) could be used for this purpose.

APPENDIX A

SYSTEMS TO MEASURE THE MOVEMENT OF THE ANCHOR

Basic Principle

The principle behind the method is simple, as can be seen in the first diagram shown at the end of this Appendix. Movement that occurs at **E** (movement of the anchor, positive if it goes upwards) translates into an equal movement at **B** (LVDT, positive if the piston plunges into the barrel), provided (a) the length of the string remains constant (if the tension does not change) and (b) the (frictionless) pulley **C**, attached to the reaction plate, does not settle. The piston of the LVDT is kept under tension via a spring.

Plate Settlement Effect

The effect of plate settlement (**C** moves down) can be addressed with the help of the second diagram.

As the length does not change (only the settlement problem is under consideration) one gets:

$$(BC)+(CD)+(DE)=(AD)+(DE), \text{ or } \quad (BC)+(CD)=(AD)$$

Coordinates of points of interest are:

Point	X	Y
A	0	0
B	$e\text{Cos}\alpha$	$e\text{Sin}\alpha$
C	$M+e\text{Cos}\alpha$	$N+e\text{Sin}\alpha$
D	$M+e\text{Cos}\alpha$	$N+e\text{Sin}\alpha+b$

Lengths of interest are:

$$B\text{-}C = (M^2 + N^2)^{\frac{1}{2}}$$

$$C\text{-}D = b \text{ (Settlement)}$$

$$A\text{-}D = [(M+e\cos\alpha)^2 + (N+e+\sin\alpha+b)^2]^{\frac{1}{2}}$$

$$b + (M^2 + N^2)^{\frac{1}{2}} = [(M+e\cos\alpha)^2 + (N+b+e\sin\alpha)^2]^{\frac{1}{2}}$$

M, N and b(plate settlement) are known so **e** (A-B movement) can be found.

Simple computations show that if **e** is positive then **b** will be positive as well.

Change in Tension Effect

The basic input data to solve this problem are:

(a) The elastic modulus of the Kevlar, $E=9*10^6$psi

(b) The thin Kevlar thread (about 0.017" in diameter) was weaved into a 4 loop rope; the cross sectional area of this rope was computed as $A = 0.00091$ in^2.

(c) The initial force set on the rope (measured in the lab) was, P = 3 lb, corresponding to a 7 inch elongation of the spring.

The change in length due to a given movement of the piston can be computed as:

$$\delta\text{Length} = \delta\sigma * \text{Rope length} / \text{Elastic modulus}.$$

Tests done at the lab indicated that a movement of 0.1 inches led to a change in load of 0.043 lb, that is to say a change in stress of $\delta\sigma = 47$ psi.

If one chooses a rope length of 15 ft (maximum value at Newberry, approximately), the corresponding change in length would be:

$$\delta\text{Length} = 0.0009 \text{ inches}$$

The above computed value is negligible compared to the settlement component.

Improved Design

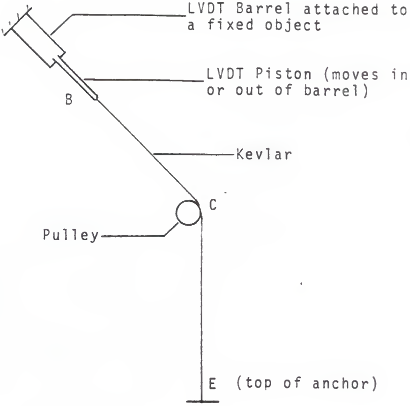
The effects described in the previous paragraphs can be circumvented if:

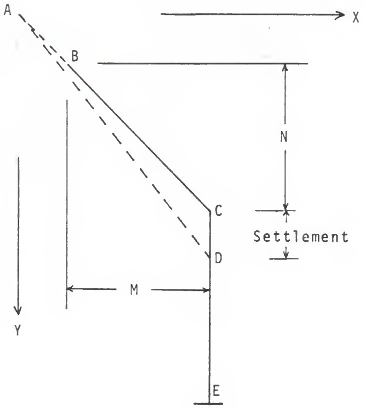
A) The Kevlar thread is kept independent of the reaction system, ie. the pulley (B) is mounted on an independent reference beam.

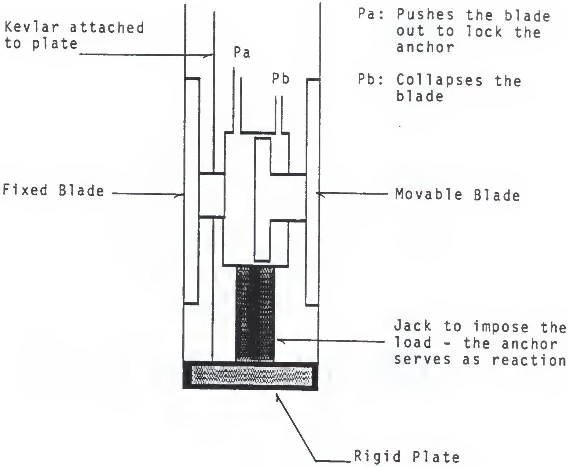
B) The tensile force used to keep the thread taut does not change; an arrangement that considers a hanging weight seemed to be appropriate.

Tip behavior

The load-settlement response of the tip could be found by means of a setup similar to the one presented at the end of this Appendix. A mechanical anchor may provide the reaction so that a jack can push a rigid metallic plate located at the bottom of the hole. The movement of the plate could be monitored by means of a system similar to the one described above.


LVDT Barrel attached to a fixed object
LVDT Piston (moves in or out of barrel)
B
Kevlar
C
Pulley
E (top of anchor)


A
X
B
N
C
Settlement
D
M
Y
E



Setup to define tip behavior

APPENDIX B
SITE EXPLORATION. SEISMIC PROSPECTION AT NEWBERRY
CALIPER RECORDS

NEWBERRY SITE

SUMMARY OF RECOVERY AND RQD

BORING #1

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 4.5	White, Shelly and granular limestone	33	10
4.5 - 9	Very Weathered Rock (recovered almost nothing)	0	0
9 - 14	Silty-Sand (Weathered Rock) (recovered a few small pieces of rock)	--	--
14 - 18.5	Shelly Limestone	31	31

BORING #2

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 9	Soft Rock - No Water Loss (used tricone)	--	--
9 - 14	" " " " " " "	--	--
14 - 18.5	Soft, Clayey Material W/Rock Pieces (specially shelly lstone.)	20	14

BORING #3

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 4	Soft, Shelly Limestone (used tricone and flushed the hole)	--	--
4 - 7	Soft, Shelly and chalky Limestone	74	40

Flushed, but 1ft of debris still remained at the bottom.

BORING #4

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 5	Soft, Shelly Rock (used tricone; set barrel & flushed but about 1 ft of sediment settled on the bottom.)	--	--
5 - 9	Soft, Shelly Limestone (recovered pieces)	37	16
9 - 14	" " " " "	38	10
	(material underneath seems to be softer per the driller		

BORING #5

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 5	Soft, Shelly (Finer) Rock (Bottom 1.5ft full of sand) -Set barrel & drilled to 7ft.	--	--
5 - 7	Soft, Shelly Rock (Used core barrel - Bottom 0.5ft full of sand)	38	~29

BORING #6

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 5.5	White, Porous, Shelly and chalky limestone. Three cores recovered	38	32
5.5 - 8.5	White, Porous, Shelly Limestone.	50	25

BORING #7

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 3.8	Shelly and chalky Limestone	54	35
3.8-8.5	" " "	47	18
8.5-13.5	" " (A few pieces recovered - the bottom 1ft of the hole is filled w/sand)	24	20

Total loss of water circulation at about 15ft. The drilling was somewhat hard (no evidence of loose sand or a cavern underneath the drilling bit; maybe a cavity exists in the side of the hole.)

13.5-16.5	Shelly and chalky Limestone	14	14
16.5-19	Shelly Limestone; Oxide Stains	85	58

BORING #8

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 4	Granular and Shelly Limestone.	31	18
4 - 9	" " "	28	16

9 - 14	" " " (Poor recovery; Loss of circulation @ 15ft)	28	15
14 - 18	Shelly Limestone	40	8
18 - 22	" "	67	35

GULF HAMMOCK SITE

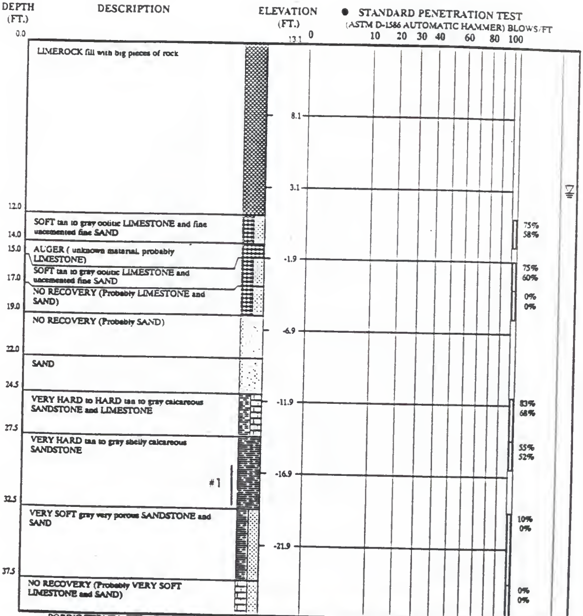
SUMMARY OF RECOVERY AND RQD

BORING #1

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 4	Dolomite, fine-grained	70	48
4 - 9	" " " (Bottom 1.5ft very weathered)	80	48
9 - 14	Dolomite, more porous, w/shells (recovered a few small pieces of rock)	76	56
14 - 17.2	Dolomite, more porous, w/shells	100	100

BORING #2

DEPTH (ft)	DECRIPTION	RECOVERY (%)	RQD (%)
0 - 3.8	Dolomite, fine-grained	71	47
3.8-8.8	Dolomite, fine-grained (Weathered @ ~6.8ft)	84	62
8.8-13.1	Dolomite, more porous	95	81
13.1-15	Dolomite, more porous	100	67



BORING TERMINATED AT 55.0 FEET

REMARKS:

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD

BORING NUMBER	PT-1
DATE DRILLED	September 19, 1990
PROJECT NUMBER	SF-4448B
PROJECT	DADELAND NORTH PARKING
PAGE 1 OF 2	DRAWN BY : J.M.A.

LAW ENGINEERING - Miami, Florida

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)	STANDARD PENETRATION TEST (ASTM D-1586 AUTOMATIC HAMMER) BLOWS/FT 0 10 20 30 40 60 80 100
	NO RECOVERY (Probably VERY SOFT LIMESTONE and SAND)		
42.5	SOFT to MEDIUM tan to white porous to very porous shelly sandy LIMESTONE with trace of uncemented sand		36% 71%
		-31.9	
46.0	MEDIUM to MODERATE HARD tan to white porous shelly sandy LIMESTONE		57% 59%
50.0		-36.9	
	MEDIUM to MODERATE HARD tan to white porous to very porous shelly sandy LIMESTONE with trace of uncemented sand		10% 9%
52.0	NO RECOVERY (Probably fine SAND)		
54.0			
55.0	NO RECOVERY (Probably LIMESTONE and SAND)	-41.9	
		-46.9	
		-51.9	
		-56.9	
		-61.9	

BORING TERMINATED AT 55.0 FEET

REMARKS:

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD

BORING NUMBER	PT-1
DATE DRILLED	September 19, 1990
PROJECT NUMBER	SF-4448B
PROJECT	DADELAND NORTH PARKING
PAGE 2 OF 2	DRAWN BY : J.M.A.

LAW ENGINEERING - Miami, Florida

DEPTH (FT.) | DESCRIPTION | ELEVATION (FT.) | ● STANDARD PENETRATION TEST (ASTM D-1586 AUTOMATIC HAMMER) BLOWS/FT

0 10 20 30 40 60 80 100

DEPTH (FT.)	DESCRIPTION	
0.0	LIMEROCK fill	
11.5	SOFT tan porous to very porous oolitic LIMESTONE and uncemented fine SAND	99% 77%
15.5	SOFT tan porous to very porous oolitic LIMESTONE and uncemented fine SAND	42% 26%
18.5	VERY SOFT tan very porous oolitic LIMESTONE and uncemented fine SAND	
19.5	SOFT tan porous to very porous oolitic LIMESTONE and uncemented fine SAND	
20.5	NO RECOVERY (Possibly very porous sandy LIMESTONE and fine SAND)	0% 0%
24.8	SOFT tan to gray very porous to porous shelly calcareous SANDSTONE and uncemented fine SAND	25% 0%
25.8	HARD gray porous to very porous slightly shelly SANDSTONE and uncemented SAND	24% 22%
30.8	SOFT to MEDIUM gray porous to very porous SANDSTONE and uncemented fine SAND	38% 29%
35.8	VERY SOFT gray very porous SANDSTONE and uncemented fine SAND	20% 0%

Elevation (FT.): 13.0, 8.0, 3.0, -2.0, -7.0, -12.0, -17.0, -22.0

BORING TERMINATED AT 60.0 FEET

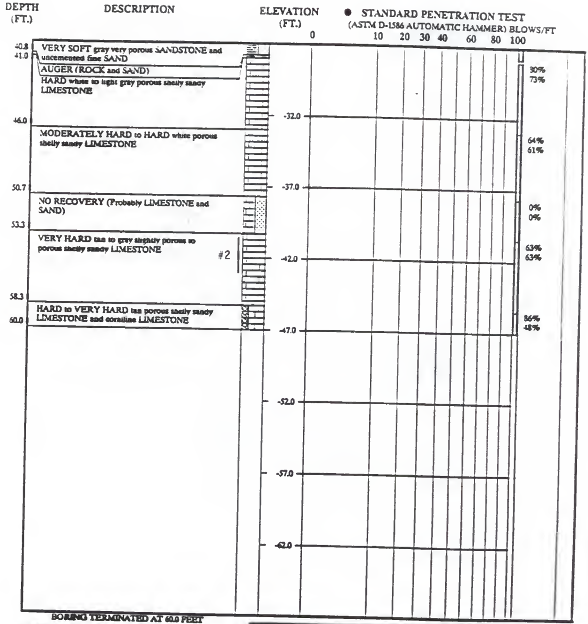
REMARKS:

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD

BORING NUMBER	PT-2
DATE DRILLED	September 25, 1990
PROJECT NUMBER	SF-4448B
PROJECT	DADELAND NORTH PARKING
PAGE 1 OF 2	DRAWN BY : J.M.A.

LAW ENGINEERING - Miami, Florida



BORING TERMINATED AT 60.0 FEET

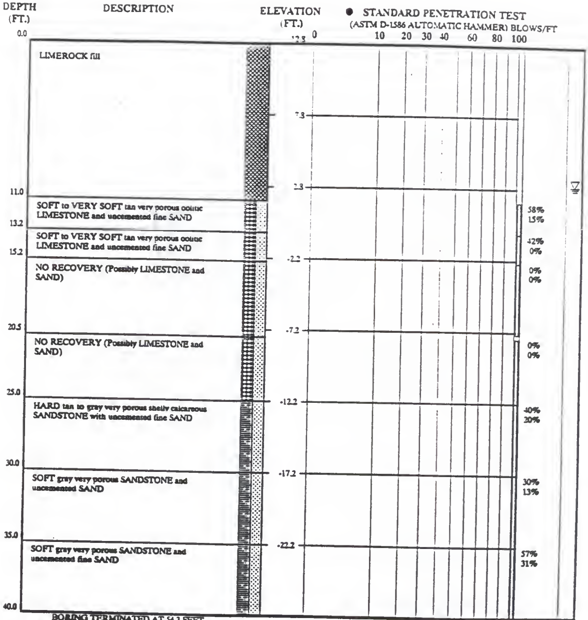
REMARKS:

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD

BORING NUMBER	PT-2
DATE DRILLED	September 25, 1990
PROJECT NUMBER	SF-4448B
PROJECT	DADELAND NORTH PARKING
PAGE 2 OF 2	DRAWN BY : J.M.A.

LAW ENGINEERING - Miami, Florida



BORING TERMINATED AT 54.3 FEET

REMARKS:

TEST BORING RECORD

BORING NUMBER	PT-3
DATE DRILLED	September 24, 1990
PROJECT NUMBER	SF-4448B
PROJECT	DADELAND NORTH PARKING
PAGE 1 OF 2	DRAWN BY: J.M.A.

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

LAW ENGINEERING - Miami, Florida

DEPTH (FT.)	DESCRIPTION	ELEVATION (FT.)
	MODERATE HARD to HARD tan to white porous shelly sandy LIMESTONE with some fine sand	
45.0	MODERATELY HARD to HARD tan to white slightly porous to porous shelly sandy LIMESTONE with some fine sand	-32.2
	#3	
49.3	VERY HARD tan to gray porous fossiliferous LIMESTONE with some uncemented fine sand	-37.2
54.3		
		-42.2
		-47.2
		-52.2
		-57.2
		-62.2

● STANDARD PENETRATION TEST (ASTM D-1586 AUTOMATIC HAMMER) BLOWS/FT

0 10 20 30 40 60 80 100

47%

96%
49%

65%
51%

BORING TERMINATED AT 54.3 FEET

REMARKS:

SEE KEY SHEET FOR EXPLANATION OF SYMBOLS AND ABBREVIATIONS USED ABOVE

TEST BORING RECORD

BORING NUMBER	PT-3
DATE DRILLED	September 24, 1990
PROJECT NUMBER	SF-4448B
PROJECT	DADELAND NORTH PARKING
PAGE 2 OF 2	DRAWN BY : J.M.A.

LAW ENGINEERING - Miami, Florida

NEWBERRY SITE

FIELD SEISMIC TESTS

May, 1990, Dr D. Woods

CROSS-HOLE #1.
HOLES #7(Source), #8(Near) AND #4(Far)

DEPTH (ft)	#8 TIME(ms) P	#8 TIME(ms) S	#4 TIME(ms) S	otS	DIFFERENTIAL Vs (ft/sec)	DIFFERENTIAL Vs Average
12	1.72	2.59	6.21	3.62	4590	
12		3.38	4.94	1.56	8330	5595
12		2.35	4.32	1.97	6600	
10		2.56	4.34	1.78	7300	7300
8		2.4	5	2.6	5000	
8		2.43	3.73			4585
8		2.41	5.3			
8			5.55	3.2	4170	
6		2.28	4.75	2.47	5260	5480
6		2.32	4.6	2.28	5700	
4		2	4.67	2.67	4870	
4		2	4.18	2.18	5960	5490
4		2.1	4.4	2.3	5650	
2		2.3	4.5	2.2	5910	5760
2		2.2	4.52	2.32	5600	

CROSS-HOLE #2.
HOLES #8(Source), #2(Near) AND #1(Far)

DEPTH (ft)	#2 TIME(ms) P	#2 TIME(ms) S	#1 TIME(ms) S	otS	DIFFERENTIAL Vs (ft/sec)	DIFFERENTIAL Vs Average
14		3.22	6.62	3.4	1642	1642
12		4.7	6.56	1.86	3002	3002

CROSS-HOLE #3.
HOLES #8(Source), #4(Near) AND #2(Far)

DEPTH (ft)	#4 TIME(ms) P	#4 TIME(ms) S	#2 TIME(ms) S	otS	DIFFERENTIAL Vs (ft/sec)	DIFFERENTIAL Vs Average
12		2.97	5.4	2.43	2403	2699
12		3.17	5.12	1.95	2995	
10	2.26	3.61	6.26	2.65	2204	
10		5.26	6.26	1	5840	3645
10		4.24	6.26	2.02	2891	

CROSS-HOLE #3a.
HOLES #8(Source), #4(Near) AND #2(Far)

DEPTH (ft)	#4 TIME(ms) P	#4 TIME(ms) S	#2 TIME(ms) P	#2 TIME(ms) S	otp	ots	DIFFER Vp (ft/sec)
8.5	2.21	3.44	3.05	4.43	0.84	0.99	6952
8.5	2.2	3.44	2.7	4.38	0.5	0.94	11680
6	2.06	3.56	3.14	4.18	1.08	0.62	5407
6	1.45	3.01	1.75	3.9	0.3	0.89	19467
6	1.15	3.01	1.75	3.9	0.6	0.89	9733
6	1.15	3.42	1.79	3.98	0.64	0.56	9125
6	0.73	1.62	1.21	2.58	0.48	0.96	12167
4	1.8	3.12	2.32	4.12	0.52	1	11231
4	1.06	3.54	1.92	4.5	0.86	0.96	6791
4	1.94	5.13	3.15	6.1	1.21	0.97	4826

CROSS-HOLE #4.
HOLES #8(Source), #4(Near) AND #1(Far)

DEPTH (ft)	#4 TIME(ms) P	#4 TIME(ms) S	#1 TIME(ms) S	otS	DIFFERENTIAL Vs (ft/sec)	DIFFERENTIAL Vs Average
10		3.64	6.2	2.56	4460	
10		5.36	4.57	1.23	9285	---
10		4.97				
8		4.3	4.25	0.05	---	
8		2.98	3.15	0.17	---	
8		3.77	4.74	1.03	11087	

CROSS-HOLE #5.
HOLES #8(Source), #1(Near) AND #7(Far)

DEPTH (ft)	#7 TIME(ms)	#1 TIME(ms)	otS	DIFFERENTIAL Vs (ft/sec)	Vs Average
16	2.23	6.81	4.58	3421	
16	2.23	5.43	3.2	4897	
16	1.1	5.19	4.09	3831	
16	1.33	5.87	4.54	3452	

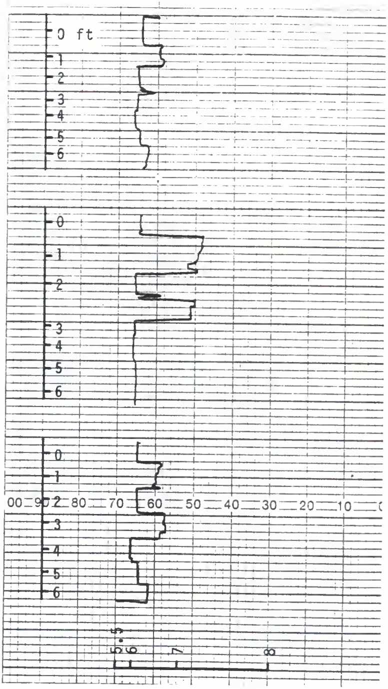
===

DOWNHOLE
Done in hole #7. Hammer set at a depth of 1 ft.

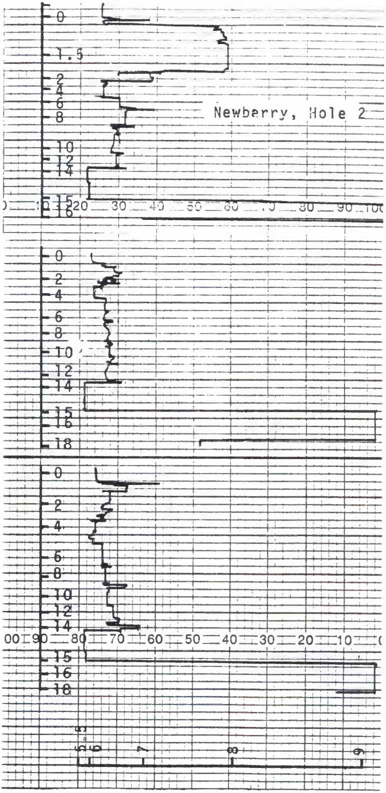
DEPTH (ft)	TIME (ms)	TIME (ms)	Vs (ft/sec)
19	2.01		8955 ?
17	1.22		13115 ?
15	0.78		17949 ?
19	1.32		13636 ?
13	0.425		28235 ?
11	0.405		24691 ?
10-19	0.905	1.93	8780
5-19	3.77	4.98	11570 ?
	-0.005	2.035	6863
15-19	1.42	1.795	10667 ?
	1.055	1.925	4598
	1.27	2.05	5128

Caliper Records

Newberry, Hole 1

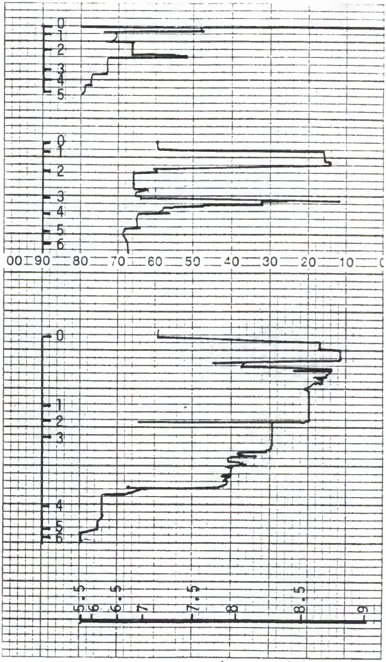


Diameter (inches)



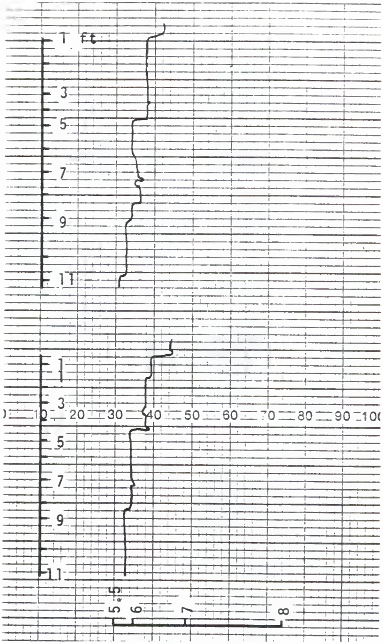
Diameter (inches)

Newberry, Hole 3

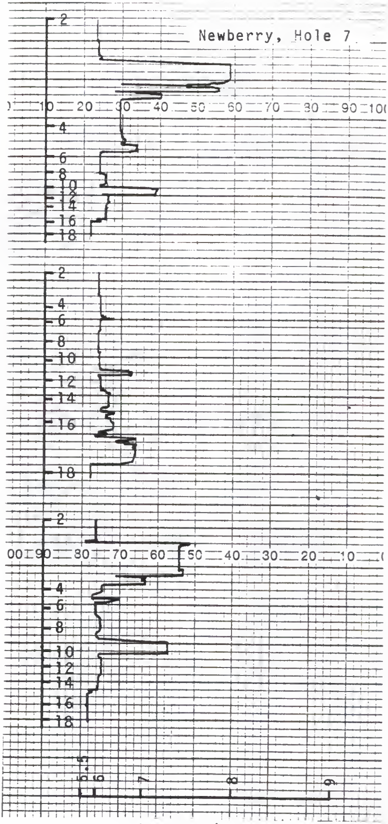


Diameter (inches)

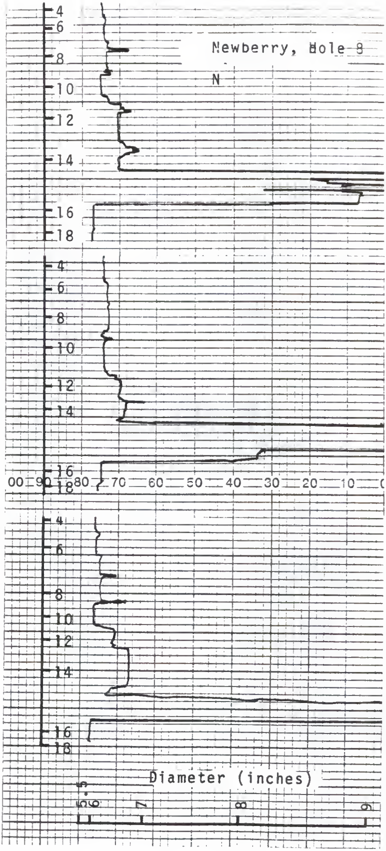
Newberry, Hole 4



Diameter (inches)

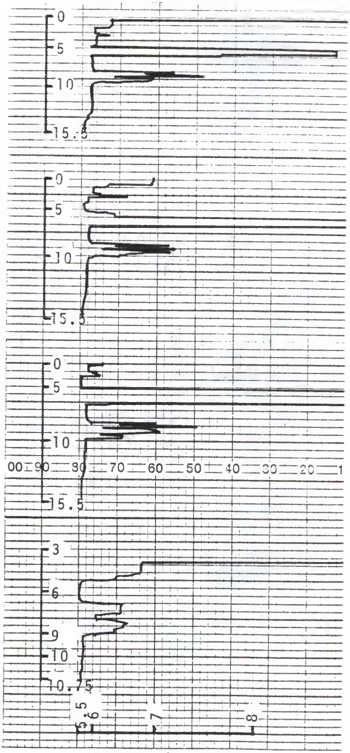


Diameter (inches)

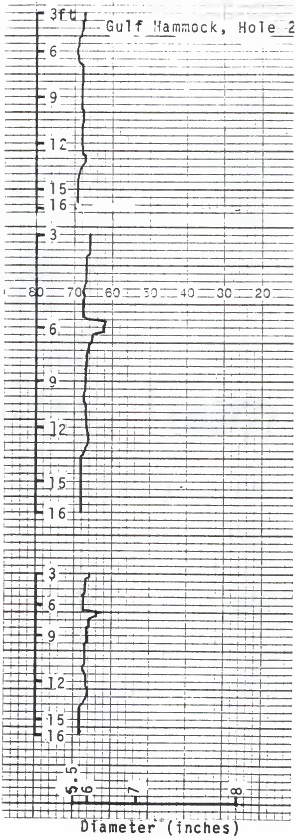


Newberry, Hole 8
N
4
6
8
10
12
14
16
18
00
90
80
70
60
50
40
30
20
10
Diameter (inches)
5.5
6
7
8
9

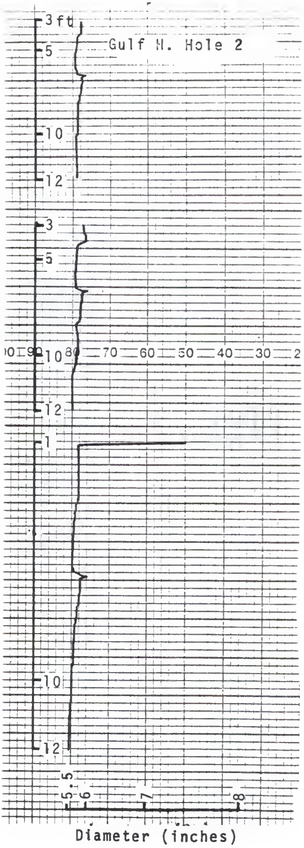
Gulf Hammock, Hole 1



Diameter (inches)



Gulf Hammock, Hole 2
3ft
6
9
12
15
16
80
70
60
50
40
30
20
5.5
6
7
8
Diameter (inches)


3 ft
Gulf H. Hole 2
5
10
12
3
5
10
12
1
10
12
5.5
6
7
8
Diameter (inches)

APPENDIX C
PULLOUT TESTS DATA

NEWBERRY

PLUG #1, LENGTH= 1ft; HOLE #5
INSTALLED AT 4.5 ft DEPTH

THE SETTL. OF THE REACTION PLATE WAS NOT RECORDED
THE ALUMINUM PLATE RESTED ON THE GROUND (NO CONCRETE PAD)
THE DIMENSIONS OF THE PLUG ARE SOMEWHAT UNRELIABLE

ANCHOR DIMENSIONS:
LENGTH= 11 in (expected 1ft)
DIAMETER= 5.8 in
TAOMAX=15.7 Tsf
TAOMAX/Z= ------
SOFTENING=0.88

PRESS.	LOAD(T)	LVDT	Z1(in)	TAO(Tsf)
-0.83	0	0.6022	0	0
-0.03	0.828157	0.42	0.045531	0.595393
1	1.89441	0.137	0.116252	1.36196
2	2.929607	-0.0614	0.165831	2.106201
3	3.964803	-0.233	0.208713	2.850442
4	5	-1.111	0.428122	3.594683
5	6.035197	-1.242	0.460859	4.338923
6	7.070393	-1.342	0.485848	5.083164
7	8.10559	-1.436	0.509339	5.827405
8	9.140787	-1.518	0.52983	6.571645
9	10.17598	-1.618	0.55482	7.315886
10	11.21118	-1.726	0.581809	8.060127
11	12.24638	-1.814	0.603799	8.804367
9.5	10.69358	1.167	0.603799	7.688006
12	13.28157	1.1151	0.616769	9.548608
13	14.31677	1.043	0.634787	10.29285
14	15.35197	0.952	0.657527	11.03709
15	16.38716	0.899	0.670772	11.78133
16	17.42236	0.8014	0.695162	12.52557
17	18.45756	0.719	0.715753	13.26981
18	19.49275	0.556	0.756486	14.01405
19	20.52795	0.401	0.79522	14.75829
20	21.56315	0.136	0.861443	15.50253
20.3	21.87371	-0.45	1.007882	15.72581
17.8	19.28571	-0.736	1.079352	13.8652

Z1 = MOVEMENT DISREGARDING SETTLEMENT

NEWBERRY

PLUG #2, LENGTH= 0.6ft; HOLE #6
DEPTH = 6.2 ft

ALUMINUM PLATE RESTED ON CONCRETE PAD
SETTL. OF PLATE NOT RECORDED
ESTIMATED SETTL. BETWEEN 0.1 AND 0.2 in

PLUG DIMENSIONS:
LENGTH= 7 in (expected 1ft)
DIAMETER= 7.5 in
TAOMAX=26.9 Tsf
TAOmax/Z = 220-330 Tsf/in; TAO/Z AT Z=0.1in= 220Tsf/in
SOFTENING=0.53

PART OF THE PLUG SEEMS TO BE IN A "CAVITY"

PRESS.	LOAD(T)	LVDT	Z1(in)	TAO(Tsf)	Z(in)
-1.06	0.000	1.822	0.000	0.000	0.000
0	1.097	1.822	0.000	0.838	0.000
1	2.133	1.822	0.000	1.629	0.000
2	3.168	1.821	0.000	2.420	0.000
3	4.203	1.770	0.013	3.211	0.006
4	5.238	1.738	0.021	4.002	0.012
5	6.273	1.714	0.027	4.792	0.017
6	7.308	1.690	0.033	5.583	0.021
7	8.344	1.650	0.043	6.374	0.029
8	9.379	1.631	0.048	7.165	0.032
9	10.414	1.609	0.053	7.956	0.036
10	11.449	1.601	0.055	8.747	0.036
11	12.484	1.582	0.060	9.537	0.040
12	13.520	1.563	0.065	10.328	0.043
13	14.555	1.550	0.068	11.119	0.044
14	15.590	1.530	0.073	11.910	0.047
15	16.625	1.484	0.084	12.701	0.057
16	17.660	1.466	0.089	13.492	0.060
17	18.696	1.451	0.093	14.282	0.062
18	19.731	1.437	0.096	15.073	0.064
19	20.766	1.423	0.100	15.864	0.066
20	21.801	1.412	0.102	16.655	0.067
21	22.836	1.403	0.105	17.446	0.067
22	23.872	1.393	0.107	18.237	0.068
23	24.907	1.378	0.111	19.027	0.070
24	25.942	1.367	0.114	19.818	0.071
25	26.977	1.349	0.118	20.609	0.074
26	28.012	1.332	0.122	21.400	0.077
27	29.048	1.323	0.125	22.191	0.077
28	30.083	1.309	0.128	22.982	0.079
29	31.118	1.294	0.132	23.772	0.081

30	32.153	1.285	0.134	24.563	0.082
31	33.188	1.274	0.137	25.354	0.083
32	34.224	1.258	0.141	26.145	0.085
33	35.259	1.245	0.144	26.936	0.087
27	29.048	0.860	0.240	22.191	0.193
25	26.977	0.700	0.280	20.609	0.236
24	25.942	0.620	0.300	19.818	0.258
24	25.942	0.400	0.355	19.818	0.313
23	24.907	0.300	0.380	19.027	0.340
21	22.836	0.100	0.430	17.446	0.393
21	22.836	-0.100	0.480	17.446	0.443
20	21.801	-0.200	0.505	16.655	0.470
17	18.696	-1.800	0.905	14.282	0.875

NEWBERRY

PLUG #3, LENGTH= 2.75ft; HOLE #1
INSTALLED AT 13.1 ft

ALUMINUM PLATE RESTED ON CONCRETE PAD
SETTL. RECORDED AT THIS SITE
SETTL. = 0.17in UNDER 60 Ton

PLUG DIMENSIONS:
LENGTH= 2.75 ft (expected 3ft)
DIAMETER= 0.5 ft
TAOMAX=21.2 Tsf
TAOmax/Z=75Tsf/in; TAO/Z AT Z=0.1in= 200 Tsf/in
SOFTENING=0.66

1st ATTEMPT

PRESS.	LOAD(T)	LVDT	Z1(in)	TAO(Tsf)	Z(in)
-1.07	0.000	1.960	0.000	0.000	0.000
0	1.108	1.960	0.000	0.256	0.000
10	11.460	1.960	0.000	2.653	0.000
20	21.812	1.813	0.037	5.049	0.021
30	32.164	1.746	0.053	7.446	0.031
40	42.516	1.660	0.075	9.842	0.045
50	52.867	1.630	0.082	12.239	0.045
60	63.219	1.516	0.111	14.635	0.066
70	73.571	1.367	0.148	17.032	0.096

2nd ATTEMPT

-1.58	0.000	1.926	0.000	0.000	0.000
0	1.636	1.926	0.000	0.379	0.000
20	22.340	1.728	0.049	5.172	0.033
40	43.043	1.695	0.058	9.964	0.025
60	63.747	1.627	0.075	14.757	0.026
80	84.451	1.405	0.130	19.550	0.066
85	89.627	1.006	0.230	20.749	0.162
87	91.698	0.517	0.352	21.228	0.282
81	85.487	-0.300	0.556	19.790	0.491
72	76.170	-0.785	0.677	17.633	0.620
70	74.099	-0.930	0.714	17.154	0.657
62	65.818	-1.500	0.856	15.237	0.806
62	65.818	-1.700	0.906	15.237	0.856
60	63.747	-1.810	0.934	14.757	0.885
60	63.747	-1.904	0.957	14.757	0.909
57	60.642	-2.057	0.995	14.038	0.949

NEWBERRY

PLUG #4, LENGTH= 1.1 ft; HOLE #1
INSTALLED AT 8.5ft

ALUMINUM PLATE RESTED ON CONCRETE PAD
SETTL. RECORDED AT THIS SITE
SETTL. = 0.17in UNDER 60 Ton

PLUG DIMENSIONS:
LENGTH= 1.12 ft (expected 2ft) PART OF THE PLUG SEEMS TO BE
DIAMETER= 0.6 ft IN A "CAVITY"
TAOMAX=30.1 Tsf
TAO max/Z=167 Tsf/in; TAO/Z AT Z=0.1in= 255 Tsf/in
SOFTENING=0.62

PRESS.	LOAD(T)	LVDT	Z1(in)	TAO(Tsf)	Z(in)
-1.4	0.000	1.974	0.000	0.000	0.000
0	1.449	1.963	0.003	0.686	0.000
2	3.520	1.865	0.027	1.667	0.008
4	5.590	1.792	0.045	2.648	0.019
6	7.660	1.741	0.058	3.629	0.026
8	9.731	1.695	0.070	4.609	0.033
10	11.801	1.654	0.080	5.590	0.039
12	13.872	1.610	0.091	6.571	0.046
15	16.977	1.577	0.099	8.042	0.050
20	22.153	1.531	0.111	10.493	0.055
25	27.329	1.487	0.122	12.945	0.061
30	32.505	1.452	0.130	15.397	0.064
35	37.681	1.416	0.139	17.849	0.068
40	42.857	1.374	0.150	20.300	0.073
45	48.033	1.320	0.163	22.752	0.082
50	53.209	1.254	0.180	25.204	0.094
55	58.385	1.039	0.234	27.656	0.143
60	63.561	0.872	0.275	30.107	0.180
37	39.752	-1.700	0.918	18.829	0.843

Z1 = MOVEMENT DISREGARDING SETTLEMENT

NEWBERRY

PLUG #5, LENGTH= 2.33 ft; HOLE #4
INSTALLED AT 5.8 ft

ALUMINUM PLATE RESTED ON CONCRETE PAD
SETTL. RECORDED AT THIS SITE
SETTL. = 0.43in UNDER 90 Ton

PLUG DIMENSIONS:
LENGTH= 2.33 ft (expected 3ft)
DIAMETER= 0.54 ft
TAOMAX=25.3 Tsf
TAOmax/Z=275Tsf/in; TAO/Z AT Z=0.1in= 240 Tsf/in
SOFTENING=0.83

1st CYCLE

PRESS.	LOAD(T)	LVDT	Z1(in)	TAO(Tsf)	Z1(in)
-1.7	0.000	1.971	0.000	0.000	0.000
0	1.760	1.968	0.001	0.445	0.000
5	6.936	1.767	0.051	1.755	0.016
10	12.112	1.590	0.095	3.064	0.042
15	17.288	1.479	0.123	4.374	0.058
20	22.464	1.400	0.143	5.683	0.070
25	27.640	1.346	0.156	6.993	0.078
30	32.816	1.309	0.165	8.302	0.085
35	37.992	1.267	0.176	9.611	0.092
40	43.168	1.233	0.184	10.921	0.097
45	48.344	1.198	0.193	12.230	0.103
50	53.520	1.164	0.202	13.540	0.109
55	58.696	1.126	0.211	14.849	0.115
60	63.872	1.098	0.218	16.159	0.120
65	69.048	1.072	0.225	17.468	0.124
70	74.224	1.049	0.230	18.778	0.127
75	79.400	1.029	0.235	20.087	0.130
80	84.576	0.996	0.244	21.397	0.135
85	89.752	0.955	0.254	22.706	0.142
88	92.857	0.938	0.258	23.492	0.145
90	94.928	0.851	0.280	24.016	0.164

2nd CYCLE

	LOAD(T)		Z(in)	TAO(Tsf)	
-1.7	0.000	1.936	0.000	0.000	0.000
0.4	2.174	1.892	0.011	0.550	0.001
10	12.112	1.647	0.072	3.064	0.019
20	22.464	1.476	0.115	5.683	0.042
30	32.816	1.364	0.143	8.302	0.062

40	43.168	1.303	0.158	10.921	0.071
50	53.520	1.263	0.168	13.540	0.075
60	63.872	1.228	0.177	16.159	0.079
70	74.224	1.196	0.185	18.778	0.081
80	84.576	1.165	0.193	21.397	0.084
90	94.928	1.125	0.203	24.016	0.087
95	100.104	1.092	0.211	25.325	0.092
85	89.752	1.060	0.219	22.706	0.107
92	96.998	1.046	0.222	24.539	0.107
79	83.540	1.038	0.224	21.135	0.116

Z1 = MOVEMENT DISREGARDING THE SETTLEMENT

GULF HAMMOCK

PLUG #1, LENGTH= 1.25 ft; HOLE #1
INSTALLED AT 12.6 ft

NEW REACTION SYSTEM WAS USED.
MEASURING SYSTEM (KEVLAR) INDEPENDENT
OF REACTION PLATE

PLUG DIMENSIONS:
LENGTH= 1.25 ft
DIAMETER= 0.48 ft
TAOMAX=13.6 Tsf
TAOmax/Z=178Tsf/in; TAO/Z AT Z=0.1in=136Tsf/in
SOFTENING=0.56

PRESS.	LOAD(T)	LVDT	Z(in)	TAO(Tsf)
-2.28	0.000	1.747	0.000	0.000
0	2.529	1.747	0.000	1.333
3	5.856	1.746	0.000	3.087
6	9.183	1.746	0.000	4.841
9	12.510	1.745	0.000	6.595
12	15.837	1.743	0.001	8.349
15	19.164	1.732	0.004	10.103
18	22.491	1.522	0.058	11.857
21	25.818	1.450	0.076	13.612
21	25.818	1.000	0.192	13.612
17.8	22.269	0.100	0.424	11.741
15.6	19.829	-0.420	0.558	10.454
11.5	15.282	-1.790	0.911	8.057
10.7	14.395	-2.000	0.966	7.589
-1.94	0.000	1.822	0.000	0.000
0	1.874	1.820	0.001	0.630
5	6.704	1.820	0.001	2.254
10	11.534	1.813	0.002	3.878
15	16.364	1.809	0.003	5.502
20	21.194	1.789	0.009	7.126
25	26.024	1.753	0.018	8.750
30	30.854	1.714	0.028	10.374
35	35.684	1.473	0.090	11.998
38.5	39.065	1.400	0.109	13.135
28.2	29.115	0.960	0.222	9.790
29	29.888	0.730	0.281	10.049
29.7	30.564	0.200	0.418	10.277
30	30.854	0.000	0.470	10.374
30	30.854	-1.500	0.856	10.374
0	1.874	-1.520	0.861	0.630

GULF HAMMOCK

PLUG #2, LENGTH= 1.96 ft; HOLE #1
INSTALLED AT 9.7 ft

NEW REACTION SYSTEM WAS USED.
MEASURING SYSTEM (KEVLAR) INDEPENDENT
OF REACTION PLATE

PLUG DIMENSIONS:
LENGTH= 1.96 ft
DIAMETER= 0.48 ft
TAOMAX=15.1 Tsf
TAOmax/Z=139Tsf/in; TAO/Z AT Z=0.1in=145Tsf/in
SOFTENING=0.79

PRESS.	LOAD(T)	LVDT	Z(in)	TAO(Tsf)
-1.94	0.000	1.822	0.000	0.000
0	2.151	1.820	0.001	0.723
5	7.696	1.820	0.001	2.588
10	13.241	1.813	0.002	4.452
15	18.786	1.809	0.003	6.317
20	24.331	1.789	0.009	8.181
25	29.876	1.753	0.018	10.046
30	35.421	1.714	0.028	11.910
35	40.966	1.473	0.090	13.774
38.5	44.848	1.400	0.109	15.080
28.2	37.307	0.960	0.222	12.544
29	38.194	0.730	0.281	12.842
29.7	38.970	0.200	0.418	13.103
30	39.303	0.000	0.470	13.215
30	39.303	-1.500	0.856	13.215
0	6.033	-1.520	0.861	2.029

GULF HAMMOCK

PLUG #3, LENGTH= 0.75 ft; HOLE #2
INSTALLED AT 13.8 ft

NEW REACTION SYSTEM WAS USED.
MEASURING SYSTEM (KEVLAR) INDEPENDENT
OF REACTION PLATE
USED MECHANICAL GAGE (0.001in)

PLUG DIMENSIONS:
LENGTH= 0.75 ft
DIAMETER= 0.48 ft
TAOMAX=14 Tsf
TAOmax/Z=140Tsf/in; TAO/Z AT Z=0.1in=140Tsf
SOFTENING=0.61

PRESS.	LOAD(T)	Z(in)	TAO(Tsf)
-2.36	0.000	0.000	0.000
0	2.617	0.000	2.300
2	4.835	0.000	4.249
4	7.053	0.000	6.198
6	9.271	0.002	8.147
8	11.489	0.004	10.096
10	13.707	0.004	12.045
12	15.925	0.100	13.994
10.3	14.040	0.100	12.337
9.6	13.264	0.170	11.655
9.2	12.044	0.200	10.583
9	11.822	0.230	10.388
8.6	11.378	0.300	9.998
7.8	10.491	0.400	9.219
7	9.604	0.600	8.439
6.5	9.049	0.700	7.952

GULF HAMMOCK

PLUG #9, LENGTH= 2 ft; HOLE #2
INSTALLED AT 9.8 fft

NEW REACTION SYSTEM WAS USED.
MEASURING SYSTEM (KEVLAR) INDEPENDENT
OF REACTION PLATE

PLUG DIMENSIONS:
LENGTH= 2 ft
DIAMETER= 0.48 ft
TAOMAX=17.7 Tsf
TAOmax/Z=160Tsf/in; TAO/Z AT Z=0.1in=176Tsf/in
SOFTENNING=0.79

PRESS.	LOAD(T)		Z(in)	TAO(Tsf)
-2.3	0.000	1.829	0.000	0.000
0	2.551	1.829	0.000	0.858
4	6.987	1.829	-0.000	2.349
8	11.423	1.829	0.000	3.841
12	15.859	1.826	0.001	5.332
16	20.295	1.822	0.002	6.824
20	24.731	1.819	0.003	8.315
25.1	30.387	1.781	0.012	10.217
30.1	35.932	1.774	0.014	12.082
35	41.366	1.768	0.016	13.909
40	46.911	1.722	0.028	15.773
42	49.129	1.717	0.029	16.519
45.2	52.678	1.400	0.111	17.712
38.6	45.358	1.180	0.167	15.251
38.2	44.915	0.990	0.216	15.102
38.1	44.804	0.730	0.283	15.065
38.5	45.247	-2.000	0.987	15.214
-2.3	0.000	0.040	0.461	0.000

GULF HAMMOCK

PLUG #5, LENGTH= 2 ft; HOLE #2
INSTALLED AT 6.3 ft (Different type of rock)

NEW REACTION SYSTEM WAS USED.
MEASURING SYSTEM (KEVLAR) INDEPENDENT
OF REACTION PLATE

PLUG DIMENSIONS:
LENGTH= 2 ft (bottom 0.5 ft in very weathered rock)
DIAMETER= 0.48 ft
TAOMAX=50 Tsf
TAOmax/Z=500 Tsf/in; TAO/Z AT Z=0.1in = 500Tsf/in
SOFTENING=0.79

PRESS.	LOAD(T)	LVDT	Z(in)	TAO(Tsf)
-2.44	0.000	1.935	0.000	0.000
0	2.706	1.934	0.000	1.189
3	6.033	1.931	0.001	2.651
6	9.360	1.927	0.002	4.112
15.1	19.452	1.927	0.002	8.546
20.2	25.108	1.925	0.003	11.031
25.2	30.653	1.923	0.003	13.467
30.2	36.198	1.923	0.003	15.903
35.2	41.743	1.922	0.004	18.340
40.2	47.288	1.921	0.004	20.776
45.2	58.378	1.920	0.004	25.648
50.2	63.923	1.915	0.005	28.084
55.2	69.468	1.913	0.006	30.521
60.2	75.013	1.911	0.006	32.957
65.2	80.558	1.909	0.007	35.393
70.2	86.103	1.903	0.008	37.829
75.2	91.648	1.898	0.010	40.265
80.2	97.193	1.889	0.012	42.702
85.2	102.738	1.886	0.013	45.138
90	108.061	1.881	0.014	47.477
95	113.606	1.822	0.029	49.913
-2.34	5.656	1.820	0.030	2.485
95	113.606	1.540	0.102	49.913

DADELAND NORTH, MIAMI
PLUG 1

PT-1, LENGTH=3 ft
INSTALLED AT 30.4 ft

THE MOVEMENT OF THE TOP OF THE
DYWIDAG BAR WAS MEASURED TO GET THE
DISPLACEMENT OF THE PLUG
THE KEVLAR SYSTEM WAS USED AS A BACKUP

PLUG DIMENSIONS:
EFFECTIVE LENGTH=1.92 ft
EFFECTIVE SIDE AREA=3 ft2
NOMINAL DIAMETER= 0.5 ft
TAOMAX=9.1 Tsf
TAOmax/Z=16.1Tsf/in; TAO/Z AT Z=0.1in= 24Tsf/in
SOFTENING=1

LOAD (Ton)	LVDT (V)	Z(UF) (in)	Z(LAW) (in)	TAO (Tsf)
0	1.877	0.000	0.000	0.000
5	1.575	0.076	0.074	1.667
10	1.308	0.142	0.125	3.333
15	1.181	0.174	0.168	5.000
20	0.802	0.269	0.253	6.667
25	0.588	0.322	0.326	8.333
27.4	-0.800	0.669	0.633	9.133
20.7	-0.800	0.669	1.021	6.900
26.3	-2.200	1.019	1.361	8.767

DADELAND NORTH, MIAMI
PLUG 2

PT-2, LENGTH=3 ft
INSTALLED AT 55.4 ft

THE MOVEMENT OF THE TOP OF THE
DYWIDAG BAR WAS MEASURED TO GET THE
DISPLACEMENT OF THE PLUG
THE KEVLAR SYSTEM WAS USED AS A BACKUP

PLUG DIMENSIONS:
EFFECTIVE LENGTH=2.5 ft
EFFECTIVE SIDE AREA=2.5 ft2
NOMINAL DIAMETER= 0.5 ft
TAOMAX=12.3 Tsf
TAOmax/Z=120Tsf/in; TAO/Z AT Z=0.1in= 100Tsf/in
SOFTENING=0.96
Note: It seems that movement at failure was about
0.1 inches

LOAD (Ton)	LVDT(V)	Z(UF) (in)	Z(LAW) (in)	TAO (Tsf)
0		0.000	0.000	0.000
5	1.789	0.000	0.050	2.000
10	1.789	0.000	0.063	4.000
15	1.789	0.000	0.072	6.000
20	1.789	0.000	0.080	8.000
25	1.781	0.002	0.099	10.000
30	1.76	0.007	0.299	12.000
30.8	0.77	0.255	0.706	12.320
29.7	-1.037	0.707	1.213	11.880
19.8	-1.156	0.736	1.586	7.920
15	-1.2	0.747	1.591	6.000
10	-1.224	0.753	1.595	4.000
5.1	-1.235	0.756	1.561	2.040
0	-1.244	0.758	1.537	0.000

DADELAND NORTH, MIAMI
PLUG 3

PT-3, LENGTH=3 ft
INSTALLED AT 47.8 ft

THE MOVEMENT OF THE TOP OF THE
DYWIDAG BAR WAS MEASURED TO GET THE
DISPLACEMENT OF THE PLUG
THE KEVLAR SYSTEM WAS USED AS A BACKUP

PLUG DIMENSIONS:
EFFECTIVE LENGTH=1.6 ft
EFFECTIVE SIDE AREA=2.9 ft2
NOMINAL DIAMETER= 0.5 ft
TAOMAX=24 Tsf
TAOmax/Z=109Tsf/in; TAO/Z AT Z=0.1in= 103 Tsf/in
SOFTENING=0.83
Note: UF displacement measurements were
used to find the T-Z slopes

LOAD (Ton)	LVDT(V)	Z(UF) (in)	Z(LAW) (in)	TAO (Tsf)
0		0.0000	0.000	0.000
5	1.97	0.0000	0.031	1.712
10	1.97	0.0000	0.055	3.425
15	1.97	0.0000	0.067	5.137
20	1.96	0.0025	0.076	6.849
25	1.96	0.0025	0.088	8.562
30	1.96	0.0025	0.101	10.274
35	1.96	0.0025	0.101	11.986
40	1.96	0.0025	0.112	13.699
45	1.96	0.0025	0.133	15.411
70	0.879		0.435	23.973
58	0.802	0.2920	0.836	19.863
60	0.213	0.4393	0.871	20.548
43.9	0.147	0.4558	1.120	15.034
27	0.135	0.4588	1.108	9.247
0	0.127	0.4608	1.039	0.000

MIAMI METROMOVER
STATION 10+30.08

PT-1, LENGTH=6.3 ft
INSTALLED AT 12.5 ft

THE MOVEMENT OF THE TOP OF THE DYWIDAG BAR WAS MEASURED TO GET THE DISPLACEMENT OF THE PLUG

PLUG DIMENSIONS:
LENGTH= 6.3 ft
DIAMETER= 0.38 ft
TAOMAX=6.2 Tsf
TAOmax/Z=26Tsf/in; TAO/Z AT Z=0.1in= 40Tsf/in
SOFTENING=0.77

TAO(Tsf)	Z(in)
0	0.000
1.08	0.015
2.16	0.029
3.24	0.080
4.32	0.107
5.4	0.140
5.94	0.159
6.2	0.239
5	0.365
4.72	0.468
4.72	0.559

MIAMI METROMOVER
STATION 15+05

PT-2, LENGTH=6.3 ft
INSTALLED AT 12.5 ft

THE MOVEMENT OF THE TOP OF THE DYWIDAG BAR WAS MEASURED TO GET THE DISPLACEMENT OF THE PLUG

PLUG DIMENSIONS:
LENGTH= 6.3 ft
DIAMETER= 0.38 ft
TAOMAX=4.6 Tsf
TAOmax/Z=20Tsf/in; TAO/Z AT Z=0.1in= 34Tsf/in
SOFTENING=0.97

TAO(Tsf)	Z(in)
0.00	0
1.07	0
2.15	0.0522
3.24	0.0879
3.76	0.1219
4.56	0.2294
4.43	0.2597
4.43	0.4729

MIAMI METROMOVER
STATION 22+73

PT-3, LENGTH=5.2 ft
INSTALLED AT 12.5 ft

THE MOVEMENT OF THE TOP OF THE DYWIDAG BAR WAS MEASURED TO GET THE DISPLACEMENT OF THE PLUG

PLUG DIMENSIONS:
LENGTH= 5.2 ft
DIAMETER= 0.38 ft
TAOMAX=6.3 Tsf
TAOmax/Z=7Tsf/in; TAO/Z AT Z=0.1in= 40Tsf/in
SOFTENING=1

TAO(Tsf)	Z(in)
0.00	0.000
1.30	0.045
2.59	0.071
3.88	0.106
4.37	0.143
5.50	0.380
6.31	0.887
6.23	0.973
6.23	1.162

MIAMI METROMOVER
STATION 31+15

PT-4, LENGTH=4.8 ft
INSTALLED AT 12.5 ft

THE MOVEMENT OF THE TOP OF THE DYWIDAG BAR WAS MEASURED TO GET THE DISPLACEMENT OF THE PLUG

PLUG DIMENSIONS:
LENGTH= 4.8 ft
DIAMETER= 0.38 ft
TAOMAX=7.2 Tsf
TAOmax/Z=32.8Tsf/in; TAO/Z AT Z=0.1in= 30Tsf/in
SOFTENING=0.87

TAO(Tsf)	Z(in)
0.00	0.000
0.71	0.009
1.42	0.062
3.55	0.115
5.32	0.150
5.67	0.168
6.73	0.191
7.18	0.220
6.20	0.309
6.20	0.412

MIAMI METROMOVER
STATION 80+86

PT-5, LENGTH=5.2 ft
INSTALLED AT 12.5 ft

THE MOVEMENT OF THE TOP OF THE DYWIDAG BAR WAS MEASURED TO GET THE DISPLACEMENT OF THE PLUG

PLUG DIMENSIONS:
LENGTH= 5.2 ft
DIAMETER= 0.38 ft
TAOMAX=4.8 Tsf
TAOmax/Z=25.1Tsf/in; TAO/Z AT Z=0.1in= 32Tsf/in
SOFTENING=0.53

TAO(Tsf)	Z(in)
0.00	0.000
0.33	0.000
0.49	0.025
1.94	0.072
3.24	0.101
4.21	0.127
4.53	0.153
4.77	0.190
2.51	0.396
2.27	0.507
2.59	0.815

MIAMI METROMOVER
STATION 89+38

PT-6, LENGTH=6.4 ft
INSTALLED AT 12.5 ft

THE MOVEMENT OF THE TOP OF THE
DYWIDAG BAR WAS MEASURED TO GET THE
DISPLACEMENT OF THE PLUG

PLUG DIMENSIONS:
LENGTH= 6.4 ft
DIAMETER= 0.38 ft
TAOMAX=6.1 Tsf
TAOmax/Z=36.1Tsf/in; TAO/Z AT Z=0.1in= 40Tsf/in
SOFTENING=0.89

TAO(Tsf)	Z(in)
0.00	0.000
1.06	0.024
2.65	0.060
3.71	0.086
4.50	0.114
5.56	0.147
6.09	0.169
5.43	0.228
5.43	0.328

APPENDIX D
SUMMARY OF LAB TEST RESULTS
PULLOUT-LAB CORRELATION (ALPHA VALUES)

NEWBERRY SITE.
SUMMARY OF LAB TESTS

SEISMIC

Vp and Vs in (ft/sec)

		IMPACT		HIGH FREQUENCY		STRENGTH		UNIT WEIGHT	L/D	
SAMPLE	DEPTH (ft)	Vp	Vs	Vp	Vs	Qu (Tsf)	Qt (Tsf)	(pcf)		CORRECTED Qu
1-1	0	7010	5483	6674	5066		30	127	0.5	
1-2		5931						121	0.3	
1-3		6884	3947				79	135	0.4	
1-4*		7516	2668	6976	4743			117	N/A	
1-5	5.5	9535	7525	11107	7689	158		134	1.8	155.9
1-6	13	9771	6364		6105		11	115	1.05	
1-7		5222	3036	7389	5956	37		120	1.2	34.4
1-8	18.5	9957	6535	9505	7273	164		132	1.3	154.4
2-1	14	9476	6733	6971	5854	199		127	0.8	168.6
2-2		9314	5959	6729	4154		17	122	0.5	
2-3		9714	5586	5880			29	140	0.3	
2-4*		4000	2400					104		
2-5	18.5	9569	6919	5189			12	114	0.4	
4-1	5	10821	5587	9431	6168	127		132	1	113.4
4-2			3144					109	0.5	
4-2*		4149	2826					139	N/A	
4-2*			2023						N/A	
4-3*			6469	7246				122	N/A	
4-4*	9		3838	6865	5198			122	N/A	

SAMPLE	DEPTH (ft)	IMPACT Vp	IMPACT Vs	HIGH FREQUENCY Vp	HIGH FREQUENCY Vs	STRENGTH Qu (Tsf)	STRENGTH Qt (Tsf)	UNIT WEIGHT (pcf)	L/D	CORRECTED Qu
6-1	0		4982	8325	5994		6	114	1.2	
6-2			6857	8797	7064	110		135	1.4	104.6
6-3			7687	6918	4750			142	0.5	
6-3*		>10000	---	8445	5630				N/A	
6-4		8644	-----					133	0.3	
6-4*		8994	-----						N/A	
6-5			2870					119	0.4	
6-6			7631	5505	4029		15	119	0.4	
6-7				9374	6794	109		132	1.6	105.8
6-12				9946	7685	212		145	1	189.3
6-13							7	108	N/A	
6-14						94			cube	
6-15	5.5					84			cube	
6-8	5.5	4515	3318	5128	3188	17		101	0.8	14.4
6-9			2794	7476	4758		7	110	0.5	
6-10			4913	6892	5635	175		138	0.6	137.0
6-11	8.5		8518				11	107	0.4	
7-1	0		3306	4679				119	0.4	
7-2			3628	6320	4927			132	0.5	
7-3			4576	5974	4866			111	0.7	
7-4			4958	6548	4753	42		109	1.3	39.8
7-5	3.8		5533	9449	7148	146		137	1.8	144.0
7-6	3.8 t		11012	8756	7039	132		140	1.1	120.1
7-7	8.5		5170	8645			36	145	0.6	
7-9	8.5-13.5		9807	8672	6255	82		156	1.4	77.8

SAMPLE	DEPTH (ft)	IMPACT Vp	IMPACT Vs	HIGH FREQUENCY Vp	HIGH FREQUENCY Vs	STRENGTH Qu (Tsf)	STRENGTH Qt (Tsf)	UNIT WEIGHT (pcf)	L/D	CORRECTED Qu
7-8	13.5-16.5		9355	8970	6890	119		129	1.3	111.5
8-1	4		8313	6280	4087		11	145	0.8	
			5846							
8-2			9067	7018	4951	83		124	0.8	70.1
8-3	9						24	134	N/A	
8-4	0 to			10598	6666	280	52	151	0.6	218.8
8-5	4			5036	3782		3	112	0.7	

* Sample tested in a direction perpendicular to the bedding plane.

GULF HAMMOCK.

SUMMARY OF LAB TESTING

SAMPLE #	DEPTH (ft)	LENGTH (ft)	SEISMIC Vp (ft/sec	SEISMIC Vs (ft/sec	STRENGTH Qu (Tsf)	STRENGTH Qt (Tsf)	GAMMA (pcf)	L/D	Qu Corrected
1-16	14	0.142				4.2			
1-15		0.283	5513	2916		3.1	129	0.9	
1-13		0.548	6084	3675	33		127	1.7	32.2
1-12		0.459	5741	4459	32		127	1.4	30.4
1-11		0.299	5971	3877		5.8	130	0.9	
1-10		0.434	6058	4344	33		129	1.3	31.1
1-9		0.372	6614	3724	54	6.0	127	1.1	49.4
1-8	9	0.217	4812	3102		3.1	126	0.7	
2-23	15	0.419	4722	3697	44		121	1.3	41.2
2-22		1.266	5412	3982	50			1.1	45.5
					56				
2-21	13	0.450	5283	3751		4.2	119	1.4	
2-20	13	0.290	6496	3593		4.8	130	0.9	
2-19		0.470	6366	4474	87		131	1.4	83.0
2-18		0.342	6487	4273	58		132	1.0	52.2
2-17		0.371	5942	4585	50		130	1.1	46.1
2-16		0.353	6471	4476	52		129	1.1	47.1
2-15		0.250	5847	3255		3.6	129	0.8	
2-14	9.9					2.2			
2-12	9.9	0.517	6616	4486	55		132	1.6	53.2
2-11	8.8	0.435	6412	4369		7.4	137	1.3	

GULF HAMMOCK.

SUMMARY OF LAB TESTING

SAMPLE #	DEPTH (ft)	LENGTH (ft)	SEISMIC Vp (ft/sec	SEISMIC Vs (ft/sec	STRENGTH Qu (Tsf)	STRENGTH Qt (Tsf)	GAMMA (pcf)	L/D	Qu Corrected
2-10	8.8	0.131	2689			0.9	124	0.4	
2-9a		0.508	6348	4439	48	3.0		1.5	46.3
2-9b						2			
2-8						7.6			
2-7c		0.661	13945	7615	677	66.0		2.0	677.1
2-7b		0.656	11576	9036	512	50.0		2.0	511.6
2-7a	4.7	0.344	9133	5733	282			1.0	254.0

DADELAND NORTH SITE.

SUMMARY OF LAB TESTS

SEISMIC

SAMPLE #	DEPTH (ft)	IMPACT Vp (ft/sec)	IMPACT Vs (ft/sec)	HIGH FREQUENCY Vp (ft/sec)	HIGH FREQUENCY Vs (ft/sec)	STRENGTH Qu (Tsf)	STRENGTH Qt (Tsf)	UNIT WEIGHT (pcf)	L/D
1-1	27.5		7939			46.4		123	2.1
1-2	|			9860	7386				
1-3	32.5	14940?					36.1	140	
2-1	53.5	11609		11240	7436				
2-2	|	13055					33.2	148	
2-3	58.0					150.7		143	1.4
3-1	46.5		7272			45.9		105	2.3
3-2	|		6519				11.8	105	
3-3	49.5		5637	8515	6199				

MIAMI METROMOVER
SUMMARY OF LAB TESTS (Miami Oolite)

STATION	DEPTH (ft)	LENGTH (ft)	Vs (ft/sec)	Vp (ft/sec)	Qu (Tsf)	Qt (Tsf)
89+38	10-15	0.333	4353	6283	11.7	2.6
80+86.84	5-10	0.333	4178	6568	8.3	
31+15	6-11	0.292	3777	6173	11.7	
22+73	6-11	0.279		6026	12.0	3.4
10+30.08	5-10	0.983	2970	5682	10.6	
	10-15	1.192	3947	6792	10.9	

SUMMARY OF ALPHA VALUES

SITE	Qu (Tsf)	ALPHA
Tampa Airport	100	0.04
" "	48	0.07
Singer Island	68	0.35
" "	68	0.12
Jax (Lstone)	16	0.63
" "	230	0.07
" "	65	0.28
" "	65	0.13
" "	65	0.02
" "	16	0.22
" "	65	0.14
" "	65	0.07
" "	63	0.05
Jax (Lstone)	48	0.3
Jax (Marl)	25	0.28
Jax (Lstone)	73	0.08
" "	63	0.15
Tampa Hospital	24	0.15
Tampa Airport	49	0.2
Miami Oolite	11.7	0.52
" "	11.7	0.62
" "	12	0.53
" "	10.6	0.58
Gulf Hammock	31	0.44
" "	40	0.38
" "	43	0.33
" "	49	0.36
" "	595	0.08
Newberry	154	0.18
" "	105	0.2
Miami Site	46.4	0.21
" "	151	0.08
" "	46	0.52

Qu = Rock unconfined (or uniaxial)
compressive strength

MCVAY'S APPROACH. COLLECTED DATA

SITE	Qu/Qt	ALPHA
Miami Oolite	4.5	0.52
" "	3.9	0.62
" "	3.5	0.53
" "	3.5	0.58
Gulf Hammock	8.7	0.44
" "	6.9	0.38
" "	10.2	0.33
" "	6.6	0.36
" "	10.2	0.08
Newberry	14	0.18
" "	17.5	0.2
Miami Site	4.5	0.08
" "	3.9	0.52
Jax (Marl)	8.5	0.18
Clearwater	10.2	0.15
" "	7	0.2
M. Dade	6.5	0.22
" "	3	0.3
Tampa	7.1	0.17
Gainesville	4.7	0.24
Ft.Lauderdale	1.63	0.37

Qu & Qt uniaxial Compressive and tensile strength of the rock, respectively.

REFERENCES

Crapps, D.K., "Design, construction and inspection of drilled shafts in limerock and limestone", Annual Meeting of Florida Section, A.S.C.E., 1986.

Donald, I.B., Chiu, H.K. and Sloan, S.W., "Theoretical analyses of rock socketed piles", Proceedings of the International Conference of Structural Foundations in Rock, Sidney, 1980, pp. 303-316.

Gupton, C. and Logan, T., "Design guidelines for drilled shafts in the weak rocks of South Florida", South Florida Annual A.S.C.E. meeting, 1984.

Hobbs, N.B. and Healy, P.R., "Piling in chalk", Publication of the Construction Industry Research and Information Association (C.I.R.I.A.), London, 1979.

Hoek, E. "Strength of jointed rock masses", Geotechnique 33, No 3, pp. 187-223.

Horvath, R. and Chae, K.J., "Long term settlement of model rock socketed piers", Canadian Geotechnical Journal, Vol. 26, 1986, pp. 573-594.

Horvath, R. and Kenney, T.C., "Shaft resistance of rock socketed piers", Symposium on Deep Foundations, A.S.C.E. National Convention, Atlanta, Georgia, October 1979, pp. 182-214.

Jaeger, J.C. and Cook, N.G. (1969), "Fundamentals of Rock Mechanics", published by Chapman and Hall Ltd and Science Paperbacks, 11 New Fetter Land, London, UK.

McMahan, B., "Drilled shaft design and construction in Florida", Independent study project, University of Florida, 1988.

McVay, M.C. and Townsend, F.C., "Design of socketed drilled shafts in limestone", IX Panamerican Conference on Soil Mechanics and Foundation Engineering, Viña del Mar, Chile, 1991 (in press).

Miller, J.A., "Hydrogeologic framework of the Floridian Aquifer system in Florida and in parts of Georgia, Alabama, and South Carolina", U.S. Geological Survey Professional Paper 1403-B. United States Printing Office, Washington, 1986.

Parra, F., Townsend, F.C., McVay, M.C., Martinez, R., "Design guidelines for shaft foundations, Final Report", submitted by the Department of Civil Engineering, University of Florida to the Florida Department of Transportation, July 1990.

Pells, P.J.N., Rowe, R.K. and Turner, R.M., "An experimental investigation into side shear force for socketed piles in sandstone", Proceedings of the International Conference of Structural Foundations in Rock, Sidney, 1980, pp. 291-302.

Prieto-Portar, L.A., "Elastic and strength properties of the calcareous rocks of Dade County, Florida", Symposium on Geotechnical properties, behavior and performance of calcareous soils, A.S.T.M. Special Publication 777, January 1981, pp. 359-381.

Puri, H.S., "Stratigraphy and zonation of the Ocala Group", Florida Geological Survey, Bulletin No. 38, 1957.

Robertson, P.K., Campanella, R.G., Gillespie, D. and Rice, A., "Seismic CPT to measure in situ shear wave velocity", Journal of Geotechnical Engineering, A.S.C.E., Vol. No 112, No 8, August, 1986, pp.791-803.

Rowe, R.K. and Armitage, H.H., "A design method for drilled piers in soft rock", Canadian Geotechnical Journal, Vol. 24, 1987, pp. 126-142.

Rowe, R.K. and Pells, P.J.N., "A theoretical study of pile-rock socket behavior", Proceedings of the International Conference of Structural Foundations in Rock, Sidney, 1980, pp. 253-264.

Schmertmann, J.H., "Report of development of a lime rock tension-shear test to guide drilled shaft foundation design for the DOT Keys bridges project", Submitted to Florida DOT and Girdler Exploration Co., Inc., Dec. 1977

Schmidt, W., Hoeinstine, N.W., Knapp, M.S., Lave, E., Ogden, G.M. and Scott, T.M., "The limestone, dolomite and coquina resources of Florida", Florida Geological Survey, Report of Investigation No. 88, 1979.

Townsend, F.C., Theos, J.F., Shields, M.D., Houssein, M., Coastal Caisson Drill Co. and Florida Department of Transportation, "Dynamic load test of drilled shaft", Report to Association of Drilled Shaft Contractors, Hugh B. William Industry Advancement Fund, March, 1991.

Williams, A.F., Donald, I.B. and Chiu, H.K., "Stress distribution in rock socketed piles", Proceedings of the International Conference of Structural Foundations in Rock, Sidney, 1980, pp. 317-325.

Williams, A.F., Johnston, I.W. and Donald, J.B., "The design of socketed piles in weak rock", Proceedings of the International Conference of Structural Foundations in Rock, Sidney, 1980, pp. 327-347.

BIOGRAPHICAL SKETCH

Fernando Parra was born a long time ago in Bogota, Colombia. He pursued his primary and secondary education at a demanding and spartan-like school. Later he entered the Universidad Nacional de Colombia where he was awarded the Civil Engineering degree in 1975. He worked as a practicing engineer and at the University for two years after which he received a Fulbright scholarship to attend the masters program at Duke University, where he was awarded the MSc degree in 1979. Nine years later, he decided to pursue the doctoral degree, so he applied to UF.

Mr. Parra has been very fortunate: his career has covered both academia and the consulting practice. Many of his former students are successful practitioners, and in addition, the companies for which he worked are setting the standards in his country.

Mr. Parra is happily married to Jacqueline, a courageous and lovely lady. Three children, Fernando, Rodrigo and Mauricio, conform the rest of the troop.

The writer intends to remain in this country and work as a consulting engineer and eventually as a professor.

I certify that I have read this study and in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Frank C. Townsend
Frank C. Townsend, Chairman
Professor of Civil Engineering

I certify that I have read this study and in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

David Bloomquist
David Bloomquist
Assistant Professor of Civil Engineering

I certify that I have read this study and in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

John L. Davidson
John L. Davidson
Professor of Civil Engineering

I certify that I have read this study and in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

Michael M. McVay
Michael M. McVay
Associate Professor of Civil Engineering

I certify that I have read this study and in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.

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This dissertation was submitted to the Graduate Faculty of the College of Engineering and to the Graduate School and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy

August 1991

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